FHWA-MA-EIS-82-02-F

Final Environmental Impact Statement and Final Section 4(f) Evaluation

Third Harbor Tunnel, Interstate 90/ Central Artery, Interstate 93

Boston, Massachusetts

Supportive Engineering Report

Federal Highway Administration
Massachusetts Department of Public Works

August, 1985

FOREWORD

Supportive Engineering Report is one of five separately bound reports which constitute the Final Environmental Impact Statement/Environmental Impact Report package for the Third Harbor Tunnel/Central Artery Project, as listed below.

1. Final Environmental Impact Statement/Report (Two Volumes)

2. Public Hearing: Synopsis of Testimony, Responses to Verbal Comments, Transcript

3. Supportive Engineering Report

4. Appendices

5. Two-Lane Tunnel/Optional Fort Point Channel Concepts

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1.0 INTRODUCTION

This report is a technical supplement to the Final Environmental Impact Statement/Report, Third Harbor Tunnel, Interstate 90/Central Artery, Interstate 93 Project in Boston. This Study includes investigations of alignment and profiles, cross-sections, methods of construction, right-of-way requirements and utilities, geology and soil characteristics, toll plazas, lighting and ventilation, communications and surveillance systems, sunken tube tunnel construction, construction cost estimates, and construction sequencing and scheduling.

The information which follows documents the results of the Supportive Engineering efforts conducted for the Preferred Alternative, Alternative 5A Modified (4-lane tunnel) evaluated in the Final EIS/EIR for the Third Harbor Tunnel I-90/Central Artery I-93 Project in Boston, Massachusetts.

2.1 DESCRIPTION

The Preferred Alternative increases traffic capacity on the Central Artery (north-south) by widening the existing facility (total length approximately 3.0 miles); cross-harbor capacity (east-west) is increased by construction of a Third Harbor Tunnel (THT) through South Boston linking the Massachusetts Turnpike/Central Artery interchange in Boston with Bird Island Flats, Logan Airport, and Route 1A in East Boston (total length approximately 3.9 miles). The project extends from a point on the Central Artery just north of the Southeast Expressway/Massachusetts Avenue interchange to a point on Interstate Route 93 in Charlestown approximately 700 feet north of the Gilmore Bridge; and from the Massachusetts Turnpike/Central Artery interchange to a point on Route 1A in East Boston in the vicinity of Prescott Street, via South Boston, Boston Harbor, and Logan Airport. The Third Harbor Tunnel alignment is also called the Seaport Alignment because it provides direct access to and from the regional highway system and the northern "seaport" sector of South Boston. Plans and profiles are presented on Figures 1 through 15 and Typical Sections are shown on Figures 16 through 37 (not included but also a possibility is a binocular steel tunnel for the cross harbor sunken tube).

A description of the Preferred Alternative by subarea follows:

South Bay/Fort Point Channel Area

In the southernmost section of the project, the Preferred Alternative meets the Southeast Expressway Upgrading project in the vicinity of the Massachusetts Avenue interchange (see Figure 1). A major interchange with the Massachusetts Turnpike/Central Artery/Southeast Expressway/Seaport Access Tunnel is provided, including a special purpose two-lane, two-way bus/high occupancy vehicle (HOV) roadway between the northbound and southbound Central Artery roadways, and bus ramps to/from the Third Harbor Tunnel. The HOV roadway and bus ramps enter the bus station level of the South Station Transportation Center (SSTC) for provision of exclusive bus transit links between Logan Airport, South Station, and points south and west of the City.

In the vicinity of this interchange, the northbound Central Artery roadway diverges towards the east from the existing northbound roadway, passes over the West Fourth Street Bridge, and then transitions down into a tunnel section passing under a new Herald Street Extension Bridge and Wye Connector. Within the tunnel, the roadway splits, with the Seaport Access Tunnel diverging to the right and the Central Artery tunnel to the left.

Near the southern end of the South Postal Annex and within the Fort Point Channel, the Central Artery tunnel also merges with a tunnel from the Massachusetts Turnpike and from Frontage Road. This four-lane tunnel is located below the bottom and along the west edge of the Channel as it passes the Gillette Company plant in South Boston. This tunnel carries all northbound Central Artery traffic; five lanes of the Dewey Square Tunnel will carry southbound Central Artery traffic.

The Seaport Access Tunnel (two-way, four-lanes plus weaving lanes) crosses the south end of the Fort Point Channel. The profile is set so the top of the tunnel box is near the bottom of the existing Channel; it crosses over the MBTA Red Line tunnel and the Central Artery northbound tunnel.

Surface roadways are also proposed to be constructed and/or modified in this area as part of this project. Existing Broadway Bridge will be replaced by a new bridge realigned slightly to the north and

designated as Herald Street Extension. Herald Street Extension (four lanes, median separated) will extend from Albany Street in Boston to West Broadway and Dorchester Avenue in South Boston. Portions of Frontage Road and Albany Street will also be reconstructed. A new two-way, four-lane relocated Dorchester Avenue will extend from Herald Street Extension to the vicinity of the present Dorchester Avenue Bridge. At this point, the southbound lanes will connect only to the South Postal Annex. Continuing northwards, a public, general purpose two-lane relocated Dorchester Avenue will extend to Summer Street, constructed as a bridge/pier structure above the proposed northbound Central Artery tunnel. The existing Dorchester Avenue, between Summer Street and Congress Street, will be reconstructed as a two-way roadway.

In addition to the major traffic movements provided from the Massachusetts Turnpike/Central Artery/Southeast Expressway/Seaport Access Tunnel interchange, the following movements will also be provided:

Off-Ramps

- Northbound Central Artery to Herald Street Extension at relocated Dorchester Avenue;
- Northbound Central Artery to Kneeland Street at Lincoln Street;
- Westbound THT to SSTC;
- Eastbound Massachusetts Turnpike to Atlantic Avenue and to SSTC; and
- Westbound Seaport Access Tunnel to Herald Street Extension.

On-Ramps

- West 4th Street to Westbound Massachusetts Turnpike;
- Albany Street to southbound Central Artery;
- Herald Street Extension to northbound Central Artery;
- Herald Street Extension to eastbound Seaport Access Tunnel;
- Frontage Road to westbound Massachusetts Turnpike; and
- Frontage Road to northbound Central Artery.

A ventilation building, approximately 100 feet high, is proposed in the South Bay area just north of Herald Street Extension.

Central Area

Generally north of High Street, the existing six-lane Central Artery will be depressed and widened to eight lanes (four lanes in each direction plus weaving lanes), in a new tunnel structure, to the vicinity of Causeway Street at North Station; it will be located principally within the existing Central Artery corridor, and will pass over the MBTA's Blue Line Tunnel at State Street (see Figure 4).

The northbound Central Artery tunnel in the Fort Point Channel will rise to cross over the MBTA Red Line tunnel at Summer Street; the top of the tunnel box will be above the low water level at a point about 400 feet south of Summer Street. The northbound Artery tunnel will then veer slightly to the west, leaving the Channel at approximately the existing bulkhead line at Russia Wharf. The four-lane tunnel will continue northerly, passing under the Boston Edison parcel, the Harbor Plaza Building, and the Hook Lobster Co. site, and will become aligned within the existing Central Artery corridor in the vicinity of High Street. South of High Street, the southbound Central Artery will operate through five lanes of the existing Dewey Square Tunnel.

The surface roadways along and crossing under the Central Artery will be rebuilt in their approximate present location, where possible. A surface arterial route connecting Atlantic Avenue (one-way northbound) and Purchase Street (one-way southbound) with Causeway Street is also proposed. The southbound surface arterial will be extended from Purchase Street to Kneeland Street, replacing the existing two-way Surface Artery from Summer Street to Kneeland Street as part of the proposed Dewey Square TSM Project (by others). U-turns will be provided at Pearl Street to Atlantic Avenue, and from Atlantic Avenue to Purchase Street and Pearl Street. A one-way eastbound connection from Oliver Street to Atlantic Avenue and to Northern Avenue will also be provided. Other U-turns are also selectively located to allow efficient traffic flow between the northbound and southbound surface arterial.

Ramps will be provided as follows:

<u>Off-Ramps</u>

- Northbound Central Artery to Surface Artery near North Street (two-lanes);
- Northbound Central Artery to Storrow Drive and Leverett Circle (two-lanes);
- Southbound Central Artery to Causeway Street;
- Southbound Central Artery and Surface Artery to the Callahan Tunnel;
- Southbound Central Artery to Purchase Street; and
- Southbound Central Artery to southbound Surface Artery at Summer Street and Beach Street.

<u>On-Ramps</u>

- Essex Street at South Street to Central Artery northbound;
- Atlantic Avenue at Northern Avenue to Central Artery northbound;
- Sumner Tunnel to Central Artery northbound and to Surface Artery;
- Causeway Street to Central Artery northbound;
- Southbound Surface Artery at New Sudbury Street to Central Artery southbound;
- Purchase Street at Congress Street to Central Artery southbound (2 lanes); and
- Lincoln Street at Essex Street to Central Artery southbound.

Four ventilation buildings have been proposed in the following areas: at Northern Avenue; at Atlantic Avenue (just north of High Street); at North Street; and along the rear of the Hoffman Building near North Station (Causeway Street). All ventilation structures are expected to be approximately 100 feet high. Ventilation buildings as located must be subjected to additional air quality analysis.

<u>North of Causeway Street Area</u>

North of Causeway Street, the depressed Central Artery emerges through a portal and transitions to a viaduct, crossing over the Charles River on two truss bridges, rejoining the existing Interstate Route 93 double-decked viaduct approximately 700 feet north of the Gilmore Bridge in Charlestown (see Figure 7).

The tie-in with the double-decked viaduct in Charlestown will involve some changes to the ramps as presently proposed for the MDPW's North Area project, but no change in the concept.

Portions of the existing viaduct ramps behind North Station are retained to accommodate the high-level ramps between the Central Artery to

the north and Storrow Drive/Leverett Circle to the west.

The ramps from the Central Artery northbound to Storrow Drive/Leverett Circle, and from Storrow Drive eastbound to the Central Artery southbound, will tunnel under the MBTA's commuter rail tracks at North Station and along the Charles River's edge. Nashua Street will be realigned from the vicinity of the Spaulding Rehabilitation Hospital, will pass over the Storrow Drive/Central Artery tunnel ramps, and will merge with the Central Artery northbound to Leverett Circle exit at-grade.

The existing Central Artery viaduct, the double-decked High-Level bridge over the Charles River, and portions of the double-decked Interstate Route 93 structure in Charlestown will be removed after the new Charles River bridges become operational. It now appears that design modifications to be made during Preliminary Design may not require construction along the Charles River's edge. The MDPW is committed to improve the Central Artery northbound grade from 6 percent to less than 5 percent. As presently configured, ramps will be provided as follows:

Off-Ramps

- o Central Artery northbound to Storrow Drive/Leverett Circle (two-lanes); and
- o Central Artery southbound to Leverett Circle/Storrow Drive (two-lanes) and to Causeway Street;

On-Ramps

- o Causeway Street to Central Artery northbound;
- o Storrow Drive/Leverett Circle to Central Artery northbound; and
- o Storrow Drive/Leverett Circle to Central Artery southbound (two-lanes).

South Boston/Seaport Access Area

After crossing the south end of the Fort Point Channel and into South Boston, the Seaport Access Tunnel traverses Gillette Company and Boston Wharf Company properties and passes under A Street, Summer Street, B Street, and Viaduct Street to an open one way toll plaza in the Commonwealth Flats area (see Figure 10). From the toll plaza area, the roadway enters a portal and curves towards the north, passing through Economic Development and Industrial Corporation (EDIC) property as it approaches Boston Harbor at the west edge of Boston Marine Industrial Park (BMIP). A Conrail freight track serving the area will also be relocated as part of this project.

After passing to the west of BMIP in South Boston, the Third Harbor Tunnel (two-way, four-lanes) crosses under Boston Harbor and directly to the Bird Island Flats area of Logan Airport, passing under the main shipping channels. The profile will not preclude the possible future deepening of the main shipping channel by the US Army Corps of Engineers.

Access to and from the Seaport Access Tunnel will be provided from Congress Street, Northern Avenue (two locations), and Summer Street. A two-way Service Road will be constructed from Northern Avenue to Congress Street near the western edge of Commonwealth Flats, and portions of Summer Street, Viaduct Street, and B Street will be reconstructed. Cross-harbor traffic will use the Congress Street/Service Road ramps to/from Northern Avenue.

Tolls will be collected on the inbound direction only. The Summer Street and Northern Avenue on-ramps to the southbound tunnel will be toll free, providing free access to this facility by all vehicles including trucks.

Ventilation buildings will be located in the vicinity of A Street; between B Street and Viaduct Street; and at the BMIP near C Street. All

ventilation structures are expected to be 50 to 75 feet high. A Massachusetts Turnpike Authority administration building will be located in Commonwealth Flats, overlooking the toll plaza.

Logan Airport/East Boston Area

At Bird Island Flats, the four-lane tunnel continues in a north-east direction, providing tunnel ramps to and from the Airport roadway system, generally south of the East Boston Memorial Stadium (see Figure 13). A ramp is also provided for northbound traffic to the Airport parking garage. Northbound traffic will continue in a tunnel, merging with a relocated outbound (egress) roadway to the east of the East Boston Memorial Stadium. This roadway to Route 1A northbound continues, requiring modifications to Route 1A, south of Bennington Street and relocation of the MBTA's Blue Line tracks south of Prescott Street. Modifications to Route 1A southbound are also provided to permit traffic to get to the southbound portion of the Third Harbor Tunnel.

This project involves significant changes to the Airport's roadway system. The existing Cross Road will be relocated slightly to the south; the Airport access road and relocated egress road will be grade-separated, passing under the Cross Road. The existing egress road will be removed, and direct connections to and from the parking garage will be constructed from Cross Road. A two-way relocated Service Road from the air cargo area near Route 1A will also be provided, intersecting with Cross Road, and thus providing connections to Bird Island Flats. A direct connection to the southbound Third Harbor Tunnel is also provided from relocated Cross Road and for traffic exiting the Bird Island Flats development.

A ventilation building approximately 100 feet high will be constructed on Bird Island Flats, where the Third Harbor Tunnel enters Airport property at the pier and bulkhead line of Boston Harbor.

3.0 PROPERTY AND UTILITIES

3.1 SUMMARY OF PROPERTIES AFFECTED

Table 1 identifies the total number of parcels affected by this alternative, including the extent of partial and full acquisition area required for the alternative. Figures 38 through 42 indicate the location of each parcel of land and title holder in the study corridor.

3.2 UTILITIES

South Bay (Figure 38)

A 36-inch x 36-inch combined sewer outfall at Congress Street and Dorchester Avenue will be extended to the Fort Point Channel on the south side of Congress Street.

A 60-inch combined sewer outfall at Summer Street and Dorchester Avenue will be extended to the Fort Point Channel on the south side of Summer Street.

A 60-inch combined sewer outfall on the easterly side of the Fort Point Channel at the Dorchester Avenue bridge will be connected to the proposed extension of the Roxbury Canal Conduit.

Force mains of 36-inch and 8-inch diameters from Massachusetts Turnpike Authority Pump House No. 7 in the South Bay will be relocated and connected to the proposed extension of the Roxbury Canal Conduit (See below).

The twin chamber (20-foot x 15.5-foot each) Roxbury Canal Conduit outfall into the Fort Point Channel will be extended from its present terminus at West Fourth Street to the vicinity of the South Postal Annex.

Existing utilities within the Boston Edison utility tunnel crossing the Fort Point Channel between Congress Street and Summer Street will be relocated onto the Congress Street bridge.

Telephone ducts located within the telephone submarine cable between Congress Street and Summer Street under the Fort Point Channel from Dorchester Avenue to Sleeper Street in South Boston will be relocated onto the Congress Street bridge.

16-inch and 24-inch water mains crossing Fort Point Channel from Dorchester Avenue at Congress Street to Northern Avenue in South Boston will be relocated onto the Congress Street bridge.

The 115,000 volt electric lines from Harrison Avenue, suspended on Broadway Bridge, crossing to South Boston at Dorchester Avenue will be relocated on the Herald Street Extension Viaduct.

The 115,000 volt electric lines from Boston Edison sub-station, crossing Fort Point Channel to South Boston near Northern Avenue will be relocated onto the new Northern Avenue bridge.

The 120-inch x 36-inch combined sewer outfall proposed by others in South Bay at Albany Street near Traveler Street will be extended to connect to the proposed extension of the Roxbury Canal Conduit.

A 30-inch intermediate pressure gas pipe crossing in the area of the Turnpike ramps and the railroad yard from Kneeland Street to Albany Street will be relocated in the same general vicinity.

The 32-inch x 54-inch East Side Interceptor combined sewer located in the vicinity of the railroad yards and crossing under the Turnpike Ramps, will be relocated in the same general vicinity.

Telephone ducts in South Bay will be relocated on the Herald Street Extension Viaduct.

A 20-inch force main, proposed for the future by others crossing Albany Street at Broadway and running parallel to the north side of the Broadway bridge to the outfall in South Bay will be extended to connect to the Roxbury Canal Conduit extension.

The Gillette Company discharge in South Boston will be extended on land along the east side of the Fort Point Channel running in a northerly direction to a new channel outfall located approximately 100 feet north of Mount Washington Avenue.

The MBTA Tower Interlock Communications Systems in South Bay will be rebuilt in the vicinity of their present locations.

The Massachusetts Turnpike Authority Pump House No. 7 in South Bay will be relocated in the same general vicinity.

Central Area (Figure 39)

The 66-inch sewer at Milk Street from the Surface Artery southbound to Atlantic Avenue will be replaced by the Milk Street sewer siphon in the same general vicinity.

The 84-inch storm drain at Milk Street from the Surface Artery southbound to Atlantic Avenue will be replaced by the Central Street drain siphon in the same general vicinity.

The East Side Interceptor at Atlantic Avenue and Northern Avenue will be relocated over the tunnel in the same general vicinity.

The existing 42-inch sewer at commercial street from Clinton Street to Cross Street will be removed and replaced by a 42-inch sewer running along the easterly side of the Central Artery to the south and connection at Milk Street and the East Side Interceptor.

The 66-inch drain at Commercial Street from Clinton Street to Cross Street will be relocated in the same general vicinity.

The 30 x 36-inch sewer at Traverse Street and North Washington Street will be connected to the New Traverse Street sewer.

A portion of the East Side Interceptor at Atlantic Avenue from Milk Street to south of India Street will be relocated in the same general vicinity.

The 60-inch storm drain at Commercial Street from Clinton Street to Altantic Avenue will be removed.

The 30 x 36 inch sewer at Hanover Street from Blackstone Street to Cross Street and the 30 x 36-inch at Sudbury Street from Blackstone Street to Cross Street will be replaced by a 36-inch sewer from Hanover Street at Blackstone Street running northerly along the westerly side of the tunnel to Sudbury Street and then crossing over the top of the Central Artery Tunnel to connect with the 66-inch sewer at Cross Street.

The 72-inch Purchase Street sewer/outfall at Oliver Street will be relocated in the same general vicinity.

The two 60-inch pipes (inverted sewer siphon) at Traverse Street from Canal Street to Haverhill Street under the Orange Line and the 102-inch sewer from Traverse Street to North Washington Street and the 57 x 60-inch sewer from North Washington Street to Cross Street will be replaced by the New Traverse Street sewer beginning at Canal Street and Crossing under the Green Line tracks and over the Orange Line tunnel and running along the easterly side of the tunnel to Sudbury Street. There it will connect with a 66-inch sewer and continue to North Street.

The 30-inch gas main at Cross Street and Commercial Street will be relocated from Clinton Street to Cross and Commercial Street.

The 30-inch gas main at Purchase Street from India Street to High Street will be relocated to the westerly side of the tunnel from India

Street to High Street.

The telephone cables at State Street from the Surface Artery southbound to Atlantic Avenue will be replaced in the same general vicinity as the existing.

The 115,000 volt electric lines at Causeway Street from Haverhill Street to Medford Street will be relocated in the same general vicinity.

The 115,000 volt electric lines at Purchase Street and Oliver Street crossing Central Artery to Boston Edison Sub-station near Harbor Plaza building will be relocated along Purchase Street, crossing Congress Street to Atlantic Avenue and then connecting to the Edison Sub-station near Harbor Plaza.

MDPW Pump Houses No. 1, No. 2, No. 4, and No. 5 will be removed and replaced by the new tunnel drainage system.

MDPW Electrical Substations will be removed and replaced by the new tunnel electrical system.

The MBTA substation at Haverhill Street south of Traverse Street may be relocated in the North Station area although recent data indicates that this site may not be affected during construction.

The Boston Edison Substation located on Atlantic Avenue may require reconfiguration due to the proximity of the Central Artery northbound tunnel.

The 16-inch LS and 16-inch HS water main, 12-inch gas main, six 5-inch electric (MBTA-AC), sixteen 5-inch electric (Boston Edison), and twelve 4-inch telephone lines in Causeway Street to be relocated from Haverhill Street to Canal Street.

North Area (Figure 40)

The West Side Interceptor at Causeway Street from Haverhill Street to Medford Street will be relocated from Causeway Street paralleling the Central Artery on the westerly side crossing under the Central Artery approximately 350 feet north of Causeway Street and then running parallel to the Leverett Circle ramps under the B & M RR Tracks and Nashua Street to the connection with the existing West Side Interceptor at Martha Way.

The Boston - Cambridge "A" cable telephone conduits at Leverett Circle will be maintained in service and supported on a temporary utility bridge in their existing location.

The twin 48-inch combined sewer (CS) siphon at Leverett Circle underpass will have to be extended for a distance of approximately 100 feet.

The river crossing structure for the 84-inch Charles River CS Marginal Conduit will be relocated approximately 35 feet closer to the river. The 48-inch and 84-inch CS inflow pipes will have to be relocated for a length of about 100 feet each. The 54-inch overflow pipe from the Lowell Street weir structure will be relocated for a length of about 150 feet to the Charles River.

A 36-inch water main running parallel to Interstate Route 93 northbound will be relocated for a length of 700 feet parallel to its existing location and outside the limits of the bridge pier foundations.

A 36-inch water main from Causeway Street to the north will be relocated parallel to proposed Ramp W-CN.

A bank of thirty-six 4-inch telephone ducts from Causeway Street to the north will be relocated parallel to proposed Ramp CS-W.

Design modifications to eliminate impacts to the Charles River's edge may require the taking of the Boston Edison Power Plant located on Nashua Street.

2-115KV pipe type electric

cables in Causeway Street from Haverhill Street to Canal Street.

South Boston (Figure 41)

The 72-inch storm drain at Mount Washington Avenue will be rebuilt as it passes over the mainline tunnel.

In 'A' Street, a 54-inch storm drain, 24-inch sanitary sewer, a 16-inch water line and 16 telephone ducts will be relocated in close proximity to their existing locations.

At Summer and Congress Streets, 115,000 volt electric lines will be relocated in the approximate vicinity of their existing locations.

A 30-inch sanitary sewer will be relocated from Trilling Way and Northern Avenue to the Massport Haul Road and then running in a southerly direction toward Summer Street.

Approximately 250 feet of 115,000 volt electric lines in the Massport Haul Road will be relocated in the same general vicinity.

Electric and telephone conduits in the Haul Road and Northern Avenue area will be relocated in the same general vicinity.

Electric and Telephone conduits in C Street between 7th and 8th Street will be relocated in the same general vicinity.

In Northern Avenue between the Massport Haul Road and 8th Street the following utilities will be relocated in the same general vicinity, 16-inch gas main, 2-16 inch water lines, and an 18-inch sewer.

East Boston (Figure 42)

A 60-inch storm drain located in the vicinity of the General Aviation Building will be relocated in the same general vicinity.

An 8-inch sanitary force main located along the easterly side of the General Aviation Building will be relocated in the same general vicinity.

A 10-inch fuel line in the vicinity of the Porter Street outfall will be relocated in an easterly direction parallel to the proposed tunnel.

A portion of the existing 10 foot x 10 foot drain located in Bird Island Flats will be rebuilt as it passes over the mainline tunnel.

The 10-foot x 12-foot Porter Street outfall will be relocated from a point just west of the Hill Air Cargo Building then running in a southerly direction along the west side of the mainline tunnel towards Bird Island Flats where it will connect with the existing twin 10-foot x 12-foot drain.

A 12-inch water main near the southerly end of the Eastern Air Freight Building will be relocated in the same general vicinity.

The 18-inch sewer located near the Hilton Hotel will be relocated in the same general vicinity.

The 8-inch fuel oil pipeline located near the Eastern Air Freight Building and the 8-inch fuel oil pipeline located near the Exxon Station and the 8-inch fuel oil pipeline near the Stadium will all be relocated close to their present locations.

A 20-inch water main and a bank of telephone ducts located near the Hertz Check-in Center will be relocated in the same general vicinity.

A 20-inch water main and a bank of telephone ducts located near the Exxon Station will be relocated in the same general vicinity.

Approximately 1200 feet of the 12-inch gas main located behind the Hilton Hotel will be relocated in the same general vicinity.

A bank of telephone and

electric ducts which runs from a point near the Hertz check-in center to a point near the American Airlines Hangar will be relocated in the same general vicinity.

A 60-inch drain which runs from a point near the Hertz Check-in Center to the American Airlines Hangar will be relocated in the same general vicinity.

A 12-inch gas main and a 20-inch water main located near the MBTA electric sub-station will be relocated in the same general vicinity.

A 20-inch water main and a bank of telephone ducts located near the Hertz Check-in Center will be relocated in the same general vicinity.

A 20-inch water main and a bank of telephone ducts located near the Exxon Station will be relocated in the same general vicinity.

Approximately 1200 feet of the 12-inch gas main located behind the Hilton Hotel will be relocated in the same general vicinity.

A bank of telephone and electric ducts which runs from a point near the Hertz Check-in Center to a point near the American Airlines Hangar will be relocated in the same general vicinity.

A 60-inch drain which runs from a point near the Hertz Check-in Center to the American Airlines Hangar will be relocated in the same general vicinity.

A 12-inch gas main and a 20-inch water main located near the MBTA electric sub-station will be relocated in the same general vicinity.

Table 1

SUMMARY OF PARCELS AFFECTED

	PARTIAL TAKINGS	COMPLETE TAKINGS	TOTAL PARCELS AFFECTED	TOTAL AREA TAKINGS
NORTH AREA	12	2	14	351,000 SF
CENTRAL AREA	2	17	19	582,000 SF
SOUTH BAY AREA	14	7	21	945,445 SF
SOUTH BOSTON AREA	15	5	20	1,478,000 SF
EAST BOSTON AREA	2	1	3	1,836,200 SF
TOTAL	45	32	77	5,192,645 SF

Note: Commonwealth of Massachusetts Parcels, Massachusetts Turnpike Authority Parcels, City of Boston Street Layouts, and takings within water bodies are not included.

4.0 CONSTRUCTION STAGING

4.1 ASSUMPTIONS

The assumptions used as criteria in planning of staged construction are as follows.

General

(1) Construction in South Bay, South Boston, Fort Point Channel, the cross Harbor Tunnel and East Boston areas will be completed prior to the start of construction of the connectors to/from the Callahan and Sumner tunnels. Temporary connectors will be provided to the South Station Transportation Center to maintain bus service during the Central Area construction period.

(2) The Central Artery northbound cut-and-cover tunnel in the Fort Point Channel could be available for construction traffic from the time it is completed to the time the depressed Central Artery is opened to general traffic.

(3) Construction of the permanent bus lanes to the South Station Transportation Center can be completed only after the depressed Central Artery is opened to traffic.

(4) Construction of I-93 NB and SB in the area north of Causeway Street (including the two new Charles River Bridges) will be timed so that its completion will occur simultaneously with the completion of the depressed Central Artery.

(5) Normal construction activities to be limited to two 10 hour shifts per day, 6 days/week.

(6) Pedestrian access and utilities will be continuously maintained via temporary crossings constructed immediately next to existing bridges, and/or cross roads.

(7) Unsuitable portions of excavated materials and all dredged materials to be wasted.

South Bay Area (Figures 43-48)

(1) The construction will be timed so that the completion of the cross Harbor Tunnel, Logan Airport, South Boston and South Bay Areas will occur simultaneously.

(2) The Central Artery will be continuously maintained as 3 lanes in each direction.

(3) Except for the new Northern Avenue Bridge, all of the bridges crossing the Fort Point Channel will be affected by the tunnel construction. To minimize traffic impacts during construction, only one bridge at a time will be reconstructed and a temporary bridge will be used in its place until the permanent structure is re-opened.

(4) The new Northern Avenue and West Fourth Street bridges will be designed and built as separate projects, and will accommodate the Third Harbor Tunnel/Depressed Central Artery project.

(5) A temporary railroad bridge ("wye connector") will be built to maintain rail service during the construction period when the existing wye connector is shut down.

(6) During construction, the same number of tracks as exist today will continuously be maintained into South Station (5) and across the Dorchester Branch railroad bridge (2) over Fort Point Channel.

(7) The Herald Street Extension Bridge will accommodate the MBTA's needs including possible commuter rail service or a reconfiguration of MBTA Red Line trackage, and will be constructed prior to removing the Broadway Bridge.

(8) Ramp connections to the Central Artery and Mass. Turnpike to be maintained at all times.

(9) Access to the South Postal Annex to be maintained at all times.

(10) The MBTA Red Line in the Fort Point Channel will be continuously in operation.

Central Area (Figures 49-59)

(1) All of the work associated with depressing the Central Artery from Dewey Square Tunnel to Causeway Street to be sub-divided into 5 major construction areas.

(2) Phase one in each of the 5 construction areas to consist of temporary and permanent relocation of utilities and roadways which cross the proposed slurry wall construction into 150' minimum block lengths to facilitate slurry wall construction.

(3) Phase two in each of the 5 construction areas to consist of constructing slurry walls, grade beams, underpinning, excavation, tunnel construction and backfill.

(4) Phase three in each of the 5 construction areas to consist of removal and disposal of the existing Central Artery Structure.

(5) Start of construction for each of the 5 construction areas will be delayed by 2 months to allow for the orderly mobilization of each of the General Contractors.

(6) During construction a minimum of 6 lanes for local surface traffic will be available at all times, between High St. and North St., where 8 lanes are presently available. Parking along the Surface Artery and Atlantic Avenue will not be permitted during construction. Under the existing elevated structure (from High St. to Causeway St.) a 40-foot wide haul road will be provided to the contractor for transport of construction materials.

(7) The existing surface artery in the area of tunnel box construction to be restored with pre-cast concrete slabs immediately after slurry wall, grade beams, and underpinning operations are complete.

(8) Parking areas under the existing Central Artery and the landscaped area adjacent to Atlantic Avenue may be used as staging and access areas for the Contractor. Demolition of existing buildings (Anelex-Charles River Buildings) to be executed immediately to provide additional staging areas to the contractors. The use of the precast surface artery decks as staging areas will not be allowed. However, the MDPW's policy to minimize impacts along the Charles River's edge will modify staging area availability.

(9) All excavation and tunnel box construction activities to be performed under the existing Surface Artery deck. Access to the below grade construction to each adjacent contract (by each of the contractors) will be continuously maintained.

(10) Construction to be phased in such a manner to:

a. Maintain 6 lanes of traffic on the elevated Central Artery during the construction of the depressed Central Artery.

b. Maintain 6 express lanes of traffic in the depressed Central Artery and reserve the elevated Central Artery as a collector-distributor road for local access.

c. After local connections are made to the depressed Artery, remove traffic from the elevated Artery and then remove the elevated structure.

(11) Connections between the existing elevated and proposed depressed Central Artery and the: Sumner and Callahan Tunnels, Atlantic Ave., and Purchase Street will be maintained at all times.

(12) A combination of 3 travel lanes in the Sumner and Callahan Tunnel will be maintained at all times during construction of local access ramps.

(13) Service on the MBTA Blue Line at State St. and the Orange Line below Haverhill St. will be maintained.

(14) Before the existing Central Artery portal at Congress Street is reconstructed, the new tunnel and relocated Dorchester Avenue in Fort Point Channel will be available for rerouting of traffic. The Third Harbor Tunnel will be available for Airport and Route 1A traffic.

(15) The Harbor Plaza Building will be underpinned and tenants will be able to remain during construction.

(16) The Congress Street northbound on-ramp will not be closed until the proposed Central Artery on-ramp from Atlantic Avenue is built.

(17) The High Street off-ramp will be removed only after the proposed Purchase Street off-ramp is functional.

(18) Atlantic Avenue (one way) will have one lane open at all times in the area of tunnel construction.

(19) During construction of the southbound Central Artery at the existing Congress Street portal, traffic will temporarily be unable to reach the existing Massachusetts Turnpike ramp at the southern end of the Dewey Square tunnel; a temporary modificaton at the Turnpike (Ramp A-B) will be used to maintain service.

North Area (Figures 60-64)

(1) Completion of the separate Central Artery North Area Project, including the connectors to the Mystic-Tobin Bridge, will precede the start of construction of the depressed Central Artery. In addition, certain modifications are assumed to be made in the North Area Project to accommodate construction of a depressed Central Artery.

(2) Completion of the North Area and depressed Artery construction will occur simultaneously.

(3) Three lanes of Interstate Route 93 (south of the Mystic-Tobin Bridge Connection) and ramp access to the Mystic-Tobin Bridge and City Square will be continuously maintained. Two lanes of I-93 (North of the Tobin Bridge Connection) will be continuously maintained.

(4) Ramp service to I-93 NB from Leverett Circle will be unavailable for one year. Ramp service from I-93 SB to Leverett Circle will be unavailable for 3 months. (Detour routes across the old Charles River Dam and Gilmore Bridge, with appropriate traffic management controls, will be instituted to serve this traffic.) However, the MDPW's policy to minimize impacts along the Charles River's edge will modify construction staging in this area.

(5) Ramp S-CN (Leverett Circle to I-93 N.B.) will not be built until the High-Level Bridge is removed.

(6) Proposed I-93 Northbound and Southbound will be constructed from the Gilmore Bridge to Causeway Street prior to removing the existing I-93 double deck structure.

(7) During construction ten tracks will continuously be maintained into North Station.

(8) Unused bridge decks resulting from modifications to I-93 will be removed.

(9) Vehicular access to the MDC Dam will be maintained. Parking will not be available during construction.

(10) The railroad spur to the Stop and Shop Bakery Building will not be available during construction.

(11) A minimum of 2 lanes through Leverett Circle to be maintained at all times.

(12) Vehicular access to the Registry of Motor Vehicles Building and the Spaulding Rehabilitation Hospital to be maintained at all times.

(13) Service on the MBTA Orange line under I-93 and MBTA Green Line at Leverett Circle will be maintained.

(14) Access through the MDC locks at the Charles River Dam will be maintained.

South Boston (Figures 65-67)

(1) R.R. access (one track) to the Army Base to be maintained at all times during construction.

(2) A minimum of 4 lanes on Summer Street to be continuously maintained during construction by means of a temporary bridge.

(3) Temporary detour routes will be constructed at A Street, B Street, and Northern Avenue.

(4) Relocated Northern Avenue from "B" Street to Atlantic Avenue will be completed by others prior to the start of the Third Harbor Tunnel construction.

(5) Suitable materials excavated during the cut-and-cover operation in South Boston to be used as fill in the South Bay Area.

Boston Harbor (Figures 65-67)

(1) Half of the 1200-foot wide shipping channel will remain open to navigation at all times, except as noted below. These activities must be coordinated with the Coast Guard Captain of the Port.

(2) During the tunnel sinking operations (approx. one tube per day) the channel should be closed to navigation. Again, these activities must be coordinated with the Coast Guard Captain of the Port.

(3) Dredging operations will be coordinated with general shipping activities and with the Coast Guard Captain of the Port.

(4) Dredging of Boston Harbor will be performed by the clamshell method. Hydraulic dredging should not be considered due to the need for the following requirements within Boston Harbor:

a) Dewatering area;
b) Spoils area with containment dike;
c) Treatment area for 490,000 C.Y;
d) Effect on shipping due to conflict with piping outlet;
e) Depth of channel may be excessive for hydraulic dredging.

East Boston (Figures 68-72)

(1) The same number of lanes that exist today on the airport roadways will be maintained at all times.

(2) Access to East Boston Memorial Stadium will be maintained at all times.

(3) Access to BIF, although disrupted during the entire construction phase, will be maintained.

(4) The existing number of lanes on the expressway ramps to and from the Airport will be maintained during construction, with temporary structures used as necessary to meet this criterion.

(5) Service on the MBTA Blue Line will not be significantly interrupted.

(6) Aircraft access to the Eastern Airlines Terminal and Hangar will be maintained, even if constructing a temporary satellite terminal and relocating the taxiway are required. (Underpinning the existing building, and further alignment refinements may render the temporary satellite terminal unnecessary.)

4.2 Proposed Construction Methods (Central Area)

Traffic management during the construction period is an essential part of this project.

Two alternatives for traffic management during construction were evaluated: a) direct traffic around the construction site on a temporary six lane roadway and provide local access to the city; or b) maintain the

existing elevated Central Artery during construction. Since the existing elevated Artery is presently providing local ramp access and traffic flows through the city, this latter plan was adopted for construction.

In order to provide for uninterrupted traffic flows on the elevated artery during constructon, it is essential that the existing structure be securely supported throughout the construction period. An independent support system is therefore recommended that will provide a fail-safe method of support to life and property during construction; a lateral support system which will retain the deep excavations; and a system which will limit ground movements adjacent to the construction.

Since the construction method for the depressed Central Artery must use the same corridor occupied by the existing elevated structure, only those construction methods which would accommodate both the existing and the proposed Artery were evaluated.

4.2.1 Rejected Construction Methods (Central Area)

Some of the various stage construction methods which were evaluated and rejected are summarized below.

A. Column Extension Method: Stage construction systems which require column extension of the existing Central Artery foundations were rejected for the following reasons:

(1) Existing pile foundation locations in relation to the proposed depressed Artery alignment would prohibit utilizing the new tunnel for the temporary movement of traffic prior to removal of the elevated structure.

(2) Exposure of the existing piles and excavation below the existing pile tips would result in lost foundation capacity and an unacceptable structural system.

(3) Proposed bracing of the existing Central Artery bents will interfere with maintaining traffic on the existing Surface Artery.

(4) Column extensions would require lateral support at ground level.

B. Truss and Caisson Method: Stage construction systems which would require a double truss system at each bent location supported by caissons were rejected for the following reasons:

(1) Caisson construction not readily adaptable to limited headroom conditions under the existing Artery.

(2) Proposed truss would interfere with maintaining traffic on the existing Surface Artery and with access between the surface streets and the elevated Artery.

C. Relieving Structure Method: Stage construction systems which require a relieving structure and excavation to take place below this structure were rejected for the following reasons:

(1) Exposure of the existing pile tips would result in lost foundation capacity and unacceptable structural system.

(2) Additional costs required to construct both the relieving structure at surface level and the tunnel roof of the depressed Central Artery.

(3) Proposed bracing of the existing Central Artery bents will interfere with maintaining traffic on the existing Surface Artery.

D. General use of Tiebacks were not considered feasible for restraining slurry walls in the Central Area and were rejected for the following reasons:

(1) Interference with adjacent structures.

(2) Extensive need for easements.

(3) Unacceptable soil conditions in certain areas.

4.2.2 Selected Construction Method (Central Area)

Figures 47 and 48 illustrate the stage construction sequence described below for the selected construction method adopted for the Central Area. This method will accommodate: the low overhead room; the existing piles for the elevated structure; buried obstructions; adjacent structures; groundwater; and vertical support of the elevated structure. However, this general underpinning concept will need to be modified as each specific area is investigated during the next design phase to accommodate clearance for both existing and proposed surface roads, utilities and ramps.

1. Existing Condition

Steel bents for the existing elevated structure are supported on pile caps or spread footings. The piles are steel "H" piles or cast-in-place concrete piles. The Surface Artery is located beneath the elevated structure.

2. Slurry Wall - Slurry Piles - Transverse Grade Beam-Surface Decking

The slurry walls will be extended into glacial till below the lowest excavation required for the depressed Artery tunnel box. This extension will be sufficient to ensure proper bearing and toe penetration to support the elevated Central Artery. Transverse concrete grade beams located on each side of the bents will be constructed between the slurry walls and will support the elevated structure between the slurry walls. They will also laterally support the top of the slurry walls. Each transverse grade beam will be additionally supported by two H-shaped concrete slurry piles at interior locations. These slurry piles will be formed by concrete walls installed by slurry trench methods. They will also be extended into the glacial till. The piles will have sufficient plan area and column stiffness such that they will not require lateral bracing as the excavation proceeds. An intermediate transverse grade beam and single slurry pile will be constructed between bents to complete the top lateral support of the slurry walls. Surface decking will be provided under the existing elevated Artery to maintain both public and construction traffic flow. The surface decking will be supported on the transverse grade beams.

3. Needle Beam - Longitudinal Grade Beam - Anchors

The pile caps for the elevated structure will be directly supported by transversely oriented steel needle beams placed beneath them. Anchors through the pile caps will secure them to the concrete-encased needle beams below. The needle beams will be supported by longitudinal concrete grade beams which will transfer the load to the transverse grade beams.

4.-5. Excavate - Pile Removal - Bracing

As excavation proceeds below the grade beams, multiple levels of steel cross bracing will be prestressed and installed. The existing Central Artery piles will then be removed.

6. Central Artery Construction

All elements of the depressed Central Artery can be built with the intermediate slurry pile supports in place. The intermediate supports will be spaced such that two 40' minimum roadways can be temporarily maintained in the tunnel.

7. Backfill - Remove Elevated Artery - Surface Grading

After appropriate connections are made at each end of the depressed Central Artery tunnel, six lanes of

through traffic from the elevated Artery may now be placed in the six temporary lanes of the depressed Artery tunnel. The top of the newly constructed tunnel can be backfilled; and the Central Artery viaduct removed.

8. Remove Intermediate Slurry Piles

After the intermediate slurry piles are removed, 8 lanes of traffic can then use the depressed Artery tunnel.

5.0 GEOLOGY AND SOILS

5.1. GENERAL

This chapter presents information relating to geology, subsurface conditions, and foundations of existing structures in the vicinity of the proposed project, followed by a summary of geotechnical engineering evaluations undertaken for the study. The study area has been divided into six sub-areas for the purpose of data collection and discussion. The sub-areas are outlined on Figure 79 and have been designated as follows:

1. South Bay including Fort Point Channel

2. Central Area

3. North Area

4. South Boston

5. Boston Harbor

6. Logan Airport (Bird Island Flats to Route 1A)

Test boring reports and drawings for existing structures are referenced to a variety of elevation data, including Boston Transit Commission Datum, USC&GS Mean Sea Level Datum of 1929 (National Geodetic Vertical Datum), Boston City Base, Mean Low Water Datum, Airport Datum, and the Massachusetts Turnpike Authority Datum. The reference datum for the Third Harbor Tunnel/Depressed Central Artery Project is the National Geodetic Vertical Datum, and conversions have been made where necessary.

5.2 DATA COLLECTION

5.2.1 General

Available reports of previous test borings completed along the project alignments were compiled. Sources of information included:

1. Haley & Aldrich, Inc. files and reports.

2. Contract documents and published reports by others relating to design studies for various completed and proposed construction projects adjacent to the project alignments.

3. The Journal of the Boston Society of Civil Engineers.

4. The archives of the following agencies:

   Massachusetts Department of Public Works;
   Massachusetts Turnpike Authority;
   Boston Water and Sewer Commission;
   Massachusetts Port Authority;
   Metropolitan District Commission;
   Massachusetts Bay Transportation Authority;
   City of Boston Public Facilities Department.

Information pertaining to the foundations of bridges and structures within the study area was collected from the following sources, in addition to those listed above:

1. Boston Public Library Archives.

2. City of Boston Building Department.

3. Plans and reports by other engineering firms.

4. Owners and occupants of various buildings.

5.2.2 Reports of Existing Test Borings and Laboratory Testing

Available reports of test borings were collected. Most of the logs were prepared by the agencies from which they were obtained, or by drilling subcontractors. In some cases, ground surface elevations were not noted on the boring logs, or, if given, the reference datum was not noted. As a result, when

interpreting some of the test boring information, assumptions were made with respect to ground surface elevations. In addition, soil descriptions are not always consistent, and determination of subsurface conditions included evaluation and interpretation of available test boring data.

Approximate locations of test borings were plotted on 1 inch = 100 feet scale plans. These locations have been estimated from available test boring location plans. The subsurface profiles (Figures 80 through 87) were constructed using selected data, usually from test borings located within 50 ft. from the centerline of the proposed structure. Records of a total of approximately 1100 test borings were assembled, of which over 150 were used to develop the profiles.

It is emphasized that the profiles represent inferred conditions only along the centerlines of proposed alignments. Significant variations in subsurface conditions occur in cross-section, especially in the vicinity of the original shoreline.

The results of laboratory soil tests performed in conjunction with previous projects were compiled. These tests were performed by Haley & Aldrich, Inc. and other geotechnical engineering firms for such projects as the filling of Bird Island Flats, the proposed relocation of Dorchester Avenue along Fort Point Channel, the proposed reconstruction of the Summer Street Bridge over Fort Point Channel, the East Side Interceptor in Fort Point Channel, Dover Street Bridge reconstruction and studies for various buildings and other structures.

5.2.3 Foundations of Existing Structures

Available foundation information for the structures within the study area were compiled. Copies of available foundation plans were obtained. A package of information was compiled, including building type and construction date, foundation and first floor type and level and other pertinent information.

Information about structures that are directly affected by the proposed project, and which require demolition or major underpinning work, is discussed in a later section.

5.3 SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

5.3.1 Subsurface Exploration Program

To assess subsurface conditions along the proposed tunnel alignment and to supplement existing information a test boring program was initiated. Work was performed by the Guild Drilling Co., Inc., of East Providence, Rhode Island, from April to May of both 1982 and 1983. Locations of the borings along or near this alignment are shown on Figure 79.

For explorations performed over water in Boston Harbor, a barge-mounted Mobil B40 rotary drill rig was utilized. Standard split-spoon sampling was generally conducted every five feet, with sample intervals adjusted for in-situ testing. Thin-wall tubes and stationary piston tube samples were taken in selected boreholes. In addition to tube samples, in-situ vane shear tests were performed in several of the boreholes. All borings over water were drilled to refusal or to a depth of approximately 85 feet below the mud-line.

The land borings ranged from 94 to 154 feet in depth. Standard split-spoon sampling of overburden soils was conducted. When encountered, rock was cored with a double-tube split inner liner core barrel using a NWD-3 diamond drill.

Observation of water levels in boreholes was made during and after completion of drilling for the recently completed test borings. Water levels measured in the boreholes are noted on the test boring logs. These observations may be affected by

the introduction of water into the borehole as part of the drilling procedure and by the rapid removal of tools from the hole at the completion of sampling.

A total of four groundwater monitoring instruments (piezometers and groundwater observation wells) were installed in overburden soils (glacial till). Water level observations are presented in Table 3.

Water level readings have been made in the observation wells, piezometers and drill holes at times and under conditions stated in the summary sheet and on the boring logs. These data have been reviewed and interpretation made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur with time and between observation points due to variations in season, rainfall, temperature, leakage into or out of underground utilities, construction activities in the area, and other factors.

Logs of recent test borings prepared by the Contractor are included in Table 2 (18 sheets) at the end of this chapter. Ground surface elevations for land borings and borehole coordinates for all borings were determined by survey crews of the Massachusetts Department of Public Works.

5.3.2 Laboratory Testing

A laboratory testing program was undertaken to aid in classifying the soils encountered during the subsurface exploration program and to determine their significant engineering properties. The testing was planned to supplement available existing data from previous studies.

Laboratory soil tests were conducted on marine clay samples recovered from 1 of the 5 test borings applicable to this alignment. These tests consisted of undrained shear strength tests and classification tests. The test results are summarized in Table 4 at the end of this chapter.

Unconfined compression, pocket penetrometer, Torvane and lab Vane tests were performed on the clay samples to determine the variation of undrained shear strength with depth. Determinations of natural water content and Atterberg limits were made to verify classifications and provide further identification of engineering properties.

5.4 GEOLOGY AND GROUNDWATER CONDITIONS

5.4.1 Regional Geology

The proposed alignment for the Third Harbor Tunnel/Depressed Central Artery lies within a structural and topographic feature known as the Boston Basin, which was created by glacial erosion of previously deposited, weak, fine-grained sedimentary rocks. The Boston Basin is located within the Boston Lowland Geographic District of the New England Physiographic Province. The lowland plain exhibits a subdued relief which is broken by many low lying ridges and hills.

Following the creation of the Boston Basin, a series of advances and retreats of a continental glacial ice sheet deposited sediments which have filled the basin as a result of an extremely complex sequence of events.

Most of the glacial deposits encountered within the Boston Basin are generally considered to be of late Wisconsin Age, products of Pleistocene glaciation. During this time period, the region was covered by thick accumulations of ice from which the glacial till deposits and associated land forms originated. After the final retreat of the glacial ice sheet, the area was inundated by a progressive rise in sea level. With the encroachment of the area by a marine environment, thick sequences of silt and clay-sized materials were deposited on top of the glacial till.

When the sea finally retreated to its present level and Boston Harbor came into being, organic silt and harbor sediment deposits formed, covering the glacio-marine clays. Throughout the region, small tidal embayments existed, accounting for the thin layer of organic deposits frequently found above the clays. Modification of the landscape by cutting and placement of fill by man throughout the basin area has developed the present topographic conditions.

Information on site geology has been derived from available boring information and publications on the geology of Boston. These sources reveal the following soil and rock sequence from the ground surface downward:

5.4.2 Site Geology

- o Miscellaneous Fill
- o Organic Deposit/Harbor Sediment
- o Marine Deposit
- o Glacio-marine Deposit
- o Fluvial/Glacio-fluvial Deposit
- o Glacial Till Deposit
- o Bedrock

The stratigraphy at the site, as interpreted from the test boring data, is shown on the subsurface profiles, Figures 80 through 87. The soil strata and bedrock elevations indicated on the profiles have been inferred from the boring information and local deviations from the inferred conditions should be expected. A detailed description of these geologic units and a discussion of the environment of deposition of these units is presented in the original DEIS/DEIR Supportive Engineering Report. A generalized description of each of the seven strata is given in the following paragraphs.

Fill. Fill was identified along the entire alignment of the proposed depressed Central Artery, except along those portions of the alignment passing beneath the Fort Point Channel, Charles River, and Boston Harbor. The fill ranges in thickness from approximately 3 to 48 ft. The fill, excluding the pavement and sidewalk sections, is generally described as brown, medium to fine sand, little fine gravel, trace silt with varying amounts of brick, glass, cobbles, concrete and wood. Other natural and man-made materials may be encountered within the fill.

Much of the land portions of the alignment in South Bay, and at Logan Airport occur over areas artificially created by the placement of miscellaneous fill over a number of years. The placement of fill began within the South Bay area. This area, south of the Dorchester Avenue Bridge, was filled with sand and gravel from Needham primarily during the period from 1858 to 1892. The fill encountered in this area generally consists of sand with varying amounts of silt, gravel, cinders, brick, wood, glass and other debris. Within the Fort Point Channel itself, fill was observed only in the area up to 300 feet north of the Broadway Bridge. The fill in this area was frequently found to be mixed with organic soils.

The history of filling at Logan Airport has been well documented starting in 1928 and continuing up to the 1970's when the area around Bird Island Flats was filled. It is anticipated that a great variety of fill materials are present in the vicinity of the Airport including granular material from borrow sources throughout eastern Massachusetts; rubble, granite blocks and concrete debris from Boston area construction projects; and hydraulic fill consisting of dredged glacio-marine clays and harbor sediments.

Organic Deposit/Harbor Sediments. The organic deposit typically consists of very soft to stiff gray to brown organic silt and peat. Also, the bottom of the present-day harbor is covered with

silt and clay sediment (harbor sediment) in varying thicknesses ranging from two to four ft.

Marine Deposit. Generally underlying the organic deposit is a stratum of marine sediment, known as the Boston Blue Clay, consisting of silty clay with occasional scattered layers or lenses of sand. Typically, the deposit consists of an upper discontinuous layer of stiff to very stiff, yellow, silty clay overlying a very soft to medium stiff, blue, silty clay.

Glacio-marine Deposit. A relatively thick deposit of glacio-marine sediment was identified at specific locations along the alignment of the proposed depressed Central Artery. Available exploration data indicate that the glacio-marine stratum is composed of a mixture of cohesive soil (silt and clay) and granular materials ranging in grain size from fine sand to cobbles. Although this material occasionally resembles glacial till in compositon, the glacio-marine soil appears to have been deposited in a near-shore marine environment by glacial meltwater streams.

Generally, the glacio-marine deposit appears to consist of a medium compact to compact, gray clayey sand and gravel with occasional layers and lenses of silt and clay. Thick sequences of this unit are found in the Central and Northern areas of the Central Artery between Station 142 to 154, Station 180 to 195 and near Station 200.

Fluvial/Glacio-fluvial Deposit. Deposits of sand and gravel occur at the site at various locations apparently as discontinuous layers and lenses. Typically, the strata are described as loose to very compact gray to brown gravelly medium to fine sand, little silt. These deposits were laid down by streams of glacial and non-glacial origin prior to, during, and after the inundation of the land area by marine water and deposition of the marine clay.

Glacial Till Deposit. The glacial till deposit is a heterogeneous mixture of soil, ranging in grain size from clay and silt to cobbles and boulders. The glacial till occurs over much of the site and directly overlies the bedrock. The absence of a till stratum at specific locations at the site is attributed to erosional processes.

Bedrock. The principal bedrock unit underlying the project site is a fine-grained, meta-sedimentary rock referred to as the Cambridge Argillite. Frequently, the argillite bedrock exhibits a weathered profile consisting of completely decomposed rock (soil-like material) in the upper approximately 20 ft. of the bedrock mass. Below this weathered zone in the bedrock, the argillite becomes progressively harder.

5.4.3 Subsurface Profiles

Figures 80 through 87 illustrate a generalized picture of subsurface conditions along the alternative alignments. These subsurface profiles were developed using previous boring information and data from the recent subsurface exploration program. In some cases the closest available information was as much as 900 feet off the alignment. In addition, information on subsurface conditions was lacking in some areas and dashed lines on the profiles indicate areas where conditions have been inferred.

South Bay and Fort Point Channel.

Subsurface conditions in these areas are illustrated in Figure 80. Typically, the sequence in these areas consists of fill over organics, underlain by clay and glacial till. In some sections (south of Summer Street for example) discontinuous lenses or strata of sand and silty sand were encountered on top of the clay unit or between the clay and the glacial till.

The clay unit in these areas is

of variable thickness, ranging from approximately 10 feet south of Summer Street to 100 feet in the vicinity of Broadway. In general, the thickness of the clay unit is influenced by the variation in the elevation of the top of the glacial till.

The surface of glacial till generally trends upward along the Fort Point Channel, from an elevation of approximately -120 at Dorchester Avenue to approximately -50 just south of Northern Avenue.

Information on the bedrock was available from West Fourth Street to the north end of the Channel. In all cases where rock was encountered, it appears to be a kaolinized (decomposed) argillite. The rock surface is fairly irregular. The top of decomposed rock ranges in elevation from approximately -75 near Summer Street to approximately -125 near Broadway in the South Bay.

Central Area (Station 90 to 175)

As inferred from interpretation of data from previous test borings by others and as indicated in Figure 81, the overburden soils along the centerline of the proposed depressed Central Artery range in thickness from approximately 33 to 110 ft. The overburden is composed of fill material; organic soil; and natural, inorganic soil of both glacial (sand, gravel) and marine (silt, clay) origin. The subsurface conditions along the alignment are complex with fill, organic soil, and interbedded strata of marine, glacio-marine, and fluvial origin overlying glacial till and bedrock. Typically, the fill ranges in thickness from approximately 5 to 40 ft. over a discontinuous layer of organic soil. The organic soil is composed of organic silt and peat and varies from approximately 4 (Station 138) to 35 ft. (Station 165) in thickness, where it occurs.

A major soil unit along the proposed alignment from approximately Station 90 to Station 110 and from Station 125 to Station 161 is a stratum of marine clay. This stratum generally has a crust of stiff to very stiff, yellow clay (about 17 ft. maximum thickness) that is underlain by very soft to soft, gray-blue silty clay with occasional seams and layers of fine sand. The marine clay is reported to be up to 80 ft. thick at Station 141. At Station 145, the clay is interstratified with a glacio-marine deposit.

The glacio-marine material is interpreted as having been deposited in a relatively shallow-water marine environment by glacial meltwater streams active at the site. Typically, the material comprising this deposit is primarily a coarse-grained granular soil. The deposit is described as a medium compact to compact, gray, clayey sand and gravel with occasional lenses of stiff, silty clay. The glacio-marine stratum was reported to be up to 63 ft. thick near Station 147 + 40.

As indicated on the geologic profile along the centerline, a stratum (approximately 3 to 13 ft. thick) of fluvial/glacio-fluvial soils occurs randomly interstratified with the marine clay and glacio-marine deposit.

A layer of glacial till (4 to 98 ft. thick) usually occurs just above the bedrock surface along the proposed alignment. At some locations, however, the glacial till has apparently been removed by erosional processes. The glacial till is composed of a heterogeneous mixture of soil grain sizes and frequently varies in density from medium compact or compact (upper till stratum) to very compact (lower till stratum). In the vicinity of High Street, between Station 110 and Station 125, the glacial till is reported to be up to 98 ft. thick. The deposit of glacial till at this location is believed to represent a buried landform referred to geologically as a drumlin.

The top of the bedrock surface, as inferred from test borings and as shown on Figure 81, appears to be at

elevation -72 or below. However, the presence of rock above the level of proposed deepest excavations cannot be ruled out, based on the available data.

As indicated on Figure 81, at least 70 ft. of soil is expected to be encountered between Station 105 to 120 along the Central Artery in the vicinity of the South Station tunnel. The overburden in this vicinity has been identified as primarily fill (3 to 18 ft. thick) and glacial till, with a thin (approximately 10 ft.) lense of marine clay occurring stratigraphically between the fill and glacial till strata near Station 110. The depth to rock in this area is unknown but is believed to be below the bottom of the proposed structure.

North Area (Station 170 to 208+50) I-93 Northbound

Total overburden thickness along the centerline of the proposed northbound alignment as revealed by available test boring data is approximately 37 to greater than 75 ft., as shown on Figure 82. Except at the Charles River, the fill forms a continuous blanket of variable thickness (5 to 35 ft.) over the underlying organic soil stratum. The thickness of the organic soil stratum ranges from 4 to 17 ft. The underlying strata consist of a complex stratification of layers and lenses of marine clay; fluvial/glacio-fluvial sand and gravel; and glacio-marine, clayey sand. The thickness of these individual deposits vary and the total thickness of this stratified sequence ranges from 32 to 52 ft.

The glacial till deposit of silty sand constitutes the most continuous naturally deposited soil stratum occurring along the profile. As shown on Figure 82, the bedrock in the area was encountered in the vicinity of Station 175. The bedrock has been identified as Cambridge Argillite. The surface of the bedrock underlying the Charles River is highly variable, ranging between approximately elevation -40 and elevation -70. Available records of test borings taken along the remainder of the proposed roadway alignment (Station 178 to 200) indicate that these borings were terminated at or above approximately elevation -60 without encountering the bedrock surface.

I-93 Southbound

The subsurface conditions along the proposed alignment of the southbound Central Artery roadway consist of approximately 65 ft. to greater than 70 ft. of soil. As shown in Figure 83, the soil units are similar in composition and stratigraphic sequence to those soil units encountered along the northbound alignment discussed above. The exception is the occurrence of a glacio-marine deposit consisting of clayey sand and gravel occurring at approximately Stations 185, 190 and 201. This deposit (glacio-marine) occurs at these three locations along the alignment as discontinuous, irregularly shaped lenses of varying thickness (reported maximum 15 ft.).

Leverett Circle Ramps

As shown on Figure 84, 50 to 80 ft. of soil overlies the surface of the bedrock along the major portion (Station 61 to 85) of the proposed alignment. These soils consist of fill, organic deposits, fluvial/glacio-fluvial deposits and marine deposits overlying glacial till and bedrock. Generally, the fill is composed of granular material with varying amounts of silt, clay, cinders, brick, and wood. The fill ranges in thickness from 15 to 45 ft. The underlying organic silt and peat stratum reaches a maximum thickness of approximately 15 ft. at Station 73+30.

Underlying the fill/organic soil strata are lenses of a marine deposit consisting of blue-gray silty clay interstratified with a granular fluvial/glacio-fluvial deposit of gray to brown sand and gravel. The combined thickness of these two soil units ranges from approximately 10 to 30 ft. A stratum (5 to 23 ft.) of

glacial till was encountered directly above the irregular surface of the argillite bedrock. Typically, the bedrock surface occurs between elevation -40 and elevation -65 along the proposed alignment between Stations 62 to 85.

South Boston (Station 37 to 92+30)

As shown on Figure 85, 70 to over 120 ft. of soil overlies bedrock along the proposed alignment in South Boston. A layer of fill forms a continuous surficial blanket 7 to 32 ft. in thickness along the alignment in South Boston. In most areas, a layer of organic soil (up to 25 ft. thick) underlies the fill.

A stratum of marine clay occurs throughout the project area. Along the alignment, it varies in thickness from 20 ft. to over 90 ft., but was generally less than 70 ft. thick. Between the Fort Point Channel and Summer Street (Sta. 37 to 57), the clay was generally 25 to 50 ft. thick. The clay directly underlies the organic silt in all but two areas where the strata are separated by relatively thin (less than 10 ft.) granular fluvial/glacio-fluvial deposits. A layer of the fluvial/glacio-fluvial deposit was also encountered beneath the clay stratum, overlying the glacial till, between Sta. 37 and 54. In other areas, the glacial till stratum was directly beneath by the clay stratum.

The glacial till stratum, which generally overlies bedrock, was encountered in borings throughout the area. Its thickness was determined in only three borings near Summer Street where it was found to be less than 10 ft. thick. Along this alignment, the top of the glacial till stratum was between El. -40 and -60 between Sta. 37 and 55. From Sta. 60 to 93, the glacial till is much lower, generally between El. -75 to -95.

Bedrock was encountered in two borings near Summer Street at El. -60 and -80. Samples of bedrock recovered from the borings indicated that approximately the top 15 ft. of the rock was completely weathered to the consistency of a very compact soil. At greater depths, the bedrock is described as moderately hard, gray, fine grained argillite.

Boston Harbor (Station 92+30 to 133+50)

Information was available on subsurface conditions in Boston Harbor along this alignment only in the berth between Pier 5 and the Boston Marine Industrial Park (formerly Piers 4, 3, 2 and 1) in South Boston, as indicated on Figure 86. In this area, bedrock rises from El. -60 near C Street to be between El. -35 to -45 throughout much of the berth. Above bedrock are thin layers of glacial till, clay and harbor sediments. Dredging for the ship berth has removed some of the soils originally present at the berth.

The variability of soil conditions beneath Boston Harbor was discussed previously. In Boston Harbor, the nearest borings to this alignment are 1000 to 2000 ft. away. This lack of nearby data precludes extrapolation of a soil profile between Stations 110 and 130.

Logan Airport (Station 133+50 to 230)

The soil profile for this alignment in the vicinity of Logan Airport is shown on Figure 87. The overburden is composed of fill, organic soil, sand deposits (fluvial/glacio-fluvial deposits), marine clay, and glacial till.

The fill stratum varies from 5 to over 35 ft. in thickness. In one area in Bird Island Flats, more than 55 ft. of fill was indicated in one of the previous borings. Underlying the fill are thin, discontinuous deposits of organic soils and peat, which in turn are underlain by sporadic deposits of sands and silty sands.

The thickness of the clay stratum is quite variable and depends

on the elevation of the top of the glacial till deposits. Throughout the Bird Island Flats area and into the Logan Airport area, (Sta. 133 to 205) the till is relatively high, being a submerged drumlin feature, and the clay layer is relatively thin, being only up to 30 ft. thick. In one area there apparently is no clay. Beyond Sta. 205, the remaining Logan Airport area is underlain by a significantly thicker clay stratum, which the previous borings indicate to be up to 110 ft. thick.

Subsurface information pertaining to the top of bedrock along the alignment in this sub-area was not available. The glacial till, however, is thought to be of substantial thickness.

5.4.4 Groundwater Conditions

Existing data on water levels reported in boreholes and observation wells compiled for this study was supplemented by readings obtained in groundwater observation wells installed in East Boston. The recent observation well data was obtained during the period April to September 1982 and April to May 1983.

Based on the available data, the following design groundwater levels are recommended for the purposes of the environmental impact study:

South Bay	+6.0 ft. NGVD
Central	+6.0 ft. NGVD
North	
Central Artery Sta. 164 to 170	+10.0 ft. NGVD
Central Artery Sta.. 170 to 208	+6.0 ft. NGVD
Leverett Circle Ramps Sta. 64 to 72	+10.0 ft. NGVD
Leverett Circle Ramps Sta. 72 to 92	+6.0 ft. NGVD
South Boston	+7.0 ft. NGVD
East Boston	+7.0 ft. NGVD

The high design groundwater level (elevation 10) existing between Station 164+25 to 170 (Central Artery - Central Area)and Station 64 to 72 Ramp CN-S) is required because the two alignments in these areas are on the Boston harbor side of the Charles River Dam and are very close to the water's edge. Elevated groundwater levels can occur due to tidal storm surges in the harbor. Groundwater levels will vary with precipitation, season, temperature, and construction activity.

5.4.5 Subsurface Historical Features

In addition to soil, rock, and groundwater conditions previously described, there are numerous subsurface features within the Central Area of potential importance to the construction. Reference should be made to Figure 88 during the following discussion.

Figure 88 was prepared from a base plan showing the configuration of Boston in 1895. Shown on the base plan are:

- o The Colonial Shoreline
- o The Barricado (out wharves)
- o The Mill Creek/Canal
- o The Old Causeway
- o 18th Century Wharf Line
- o 19th Century Wharf Line
- o Atlantic Avenue Elevated Footings
- o Union Freight Railroad

The original shoreline indicates the configuration of the Shawmut Peninsula as it existed during the 1640's. Subsequent filling and development has radically altered the topography and configuration of the

shoreline, but several features important to the study date from this time. The North Cove became the Mill Pond when a causeway was constructed approximately along the trace of today's Causeway Street. In 1643, the Mill Creek was excavated across the neck of land from Mill Pond to Town Cove, effectively dividing the North End from the rest of Boston. Mill Creek provided an inlet for tidal water for operating mills located on Mill Pond. At high tide, the Creek formed a barrier of water between the two parts of the City, and at low tide, the area was wet and swampy.

The Town Cove area (the region around today's Quincy Market and Faneuil Hall) was the site of the first wharf structures. During the late 1600's and 1700's, a series of timber wharves was built along the waterfront. The most significant of these wharves was Long Wharf, which was originally constructed during the period 1710 to 1712. Numerous other wharves were constructed along the waterfront between Fort Hill and the North End. Construction details are lacking, but it is likely that these wharves were built of timber planks, supported on timber piles driven into the marine deposits under the organic soils which covered the harbor bottom.

In 1673, a wall of wood and granite blocks was constructed across the Town Cove from Fort Hill to Skarlet's Wharf (North End waterfront). Remnants of this structure may survive, buried beneath present streets and structures.

With an increase in importance of the City as a port, a new generation of wharves was constructed in the 19th century. During this period, fill was placed around the 18th century wharves; Rowe's, India, Central, and City Wharves were constructed; and Long Wharf was rebuilt. Granite block walls were commonly sunk into the harbor bottom or supported on piles, with piles driven alongside to reinforce the granite blocks. Fill was placed inside the border walls and wharf houses, some of which were pile-supported, were built on the wharves.

In the early 1800's, the Mill Pond was filled, and the current street pattern in this area near North Station was laid out. By that time, Mill Creek had been upgraded to a canal and lined with granite blocks. Barges loaded with cargo used the canal, part of the Middlesex Canal System, to gain access to the seaport. In 1833, the canal was filled in and the existing Blackstone Street was laid out.

During the 20th century, two means of transportation of importance to the study area were in operation. The Union Freight Railroad, which ran along old Atlantic Avenue, transported freight to the waterfront area. The Atlantic Avenue Elevated transported passengers on an above-grade structure which was also located along Atlantic Avenue. This structure was supported on eight ft. square pile-supported concrete footings, which remain in place today.

Several of these historical features may have an impact on the construction of the Depressed Artery:

o In the Fort Hill area, remnants of the Barricado (out wharves), buried footings from the Atlantic Avenue Elevated, and vestiges of the Union Freight Railroad, as well as buried 18th century timber wharves and associated piles and possibly stone seawalls may be encountered.

o The section between Batterymarch Street and Commercial Street is an area where remnants of 19th century wharves constructed using granite block seawalls should be anticipated. Both timber piles and large granite blocks were probably buried during filling activities which took place during the late 19th and early 20th centuries.

o Between Commercial Street and North Street, remains of 18th century wharves may be encountered.

- In the area where Blackstone Street parallels the existing Artery structure, remnants of the Mill Creek canal may still be in place, buried by filling in 1833. The original Mill Creek was lined with granite blocks, which may or may not have been removed when the canal was filled in.

- Near Causeway Street, 19th century wharf structures may be encountered. Details of the construction of these wharves are not available.

- In addition to the foregoing, foundations of many buildings which previously occupied the proposed depressed alignment probably still remain.

In summary, numerous subsurface features dating throughout Boston's history are believed present along the alignment. Some of these features are anticipated to cause interferences to the construction. Subsurface explorations will be required in later design to further evaluate the location and nature of some of these structures.

5.5 GEOTECHNICAL ENGINEERING STUDIES

5.5.1 General

This section summarizes the results of geotechnical engineering studies undertaken for the Third Harbor Tunnel/Depressed Central Artery Environmental Impact Study. These evaluations and recommendations are preliminary in nature and serve as guidelines and criteria for assessing project feasibility, design concepts, environmental impacts, and cost estimates.

Geotechnical engineering studies were based on the subsurface information reported herein and on the preliminary structure layout and design. The coverage of subsurface information was generally satisfactory for the purpose of the study. However, the limited data available on the location of bedrock in the Boston Harbor and Bird Island Flats areas restricted studies relative to deep, rock tunneling and deep open-cut excavation. Also, the limited available data on certain characteristics of the bedrock restricted studies relative to the requirements for and affects of groundwater pumping from the rock. Information on engineering soil properties, based on previous laboratory testing, was not available in certain areas and limited the accuracy to which ground settlement and other estimates of soil behavior could be made. Available data from previous construction records was helpful in certain evaluations.

5.5.2 South Bay (Including Fort Point Channel to Congress Street)

General

This section is the portion of the project in the South Bay Area including the Fort Point Channel north to Congress Street. The Central Artery Northbound and South Boston tunnels extend north and east from South Bay as various ramps merge from connections to the Central Artery, Massachusetts Turnpike, the proposed South Station Transportation Center, and local streets. Project design must consider four existing bridges, the Wye Connector bridge which is presently under construction, the South Postal Annex Facility, railroad tracks and facilities, and the Red Line subway tunnels.

A complex stage-construction plan is necessary to accomplish the work and maintain in operation essential railroad lines and roadways.

Cut-and-cover type construction is planned throughout the South Bay Area for ramps and mainline tunnel structures. There will be several uncovered "boat" sections.

Filling of South Bay

Plans call for filling the South Bay area to El. 10, the typical adjacent ground level. Fill will not extend out into the Fort Point

Channel. The required fill thickness ranges from about 10 to 25 ft., and is typically 17 ft. The Roxbury Canal Conduit, which presently discharges into the South Bay, would be extended through the filled area. Settlements will necessitate pile foundation support for new structures. Drag loads and settlement will necessitate replacement of existing bridge foundations.

The South Bay is underlain by compressible soils consisting of an approximately 5 to 15-foot thick layer of soft organic silt underlain by about 90 to over 100 feet of silty clay. Fill placement will cause consolidation and settlement of the organic and clay soils. Long-term settlement of the clay stratum alone may range in magnitude from about 6 inches to as much as 2 to 3 feet, depending on the stratum thickness, its engineering properties, and its location relative to the filled area.

The major primary consolidation settlement is expected to be essentially complete on the order of ten years after filling. Thereafter, secondary consolidation settlement will occur at a reduced time rate for years afterward. Secondary consolidation settlements may be in the order of 5 to 10 inches over a 50-year period.

If left in place, the organic soils will settle under the added fill load. Settlements of the organic soils are expected to be in the order of 2 to 4 feet. Most of these settlements will occur during the construction period.

The use of a surcharge fill with deeper vertical drains to accelerate settlements is not considered feasible in view of stage construction requirements.

The plan limits of fill placement should be restricted adjacent to existing bridges that are to remain in service during the first stages of construction, in order to avoid excessive settlements and lateral deflections of the structures. The toes of embankments should be at least 10 ft. from the existing bridge foundations.

The soft and weak organic soils which underlie the South Bay may undergo significant lateral movements during uncontrolled fill placement. Excessive displacements or mud waves should be prevented in areas adjacent to foundations of bridges that are to remain during the first stage of construction. Therefore, prior to filling next to bridges, the organic materials should be excavated to the surface of the clay layer and replaced with granular fill placed "in-the-wet". Underwater excavation slopes should be no steeper than 1H:1V. The width of the bottom of excavation should be approximately 25 ft. Remaining fill should be placed first in areas closest to bridges to be protected.

It is suggested that during final design, the feasibility of a pile-supported deck be investigated as an alternative to the proposed filling of South Bay, although there are some disadvantages to this approach, as discussed below. Because there would be no filling, this approach would prevent settlements of the clay and organic soils, and new structures could then be founded on soil-supported mats rather than piles. In filling the South Bay, the weight of earth above the deeper tunnels will contribute substantially to the foundation loads to be carried on the piles.

Portions of the decking could be supported on piers bearing directly on the tunnels in and adjacent to the South Bay, and on the Roxbury Canal Conduit Extension. Piles driven to support temporarily relocated structures might also be incorporated into support for a deck. Decking the South Bay may also permit some portions of foundations of the Wye Connector and other existing structures to be saved and reused; they may otherwise have to be abandoned due to the excessive

settlements that may result from filling of South Bay.

Some disadvantages of a decking scheme include costs associated with the structure and its pile foundations, and cost and time involved with more expensive over water construction. Filling beneath the deck would have to be prohibited to avoid causing settlement of soil-supported structures and downdrag on piles. Also, because of the tidal fluctuation of the Fort Point Channel, and the open area under the deck, debris and other undesirable materials will also accummulate under the deck, possibly causing stagnant conditions, odors, etc.

Foundations for Structures

Clay settlements due to filling preclude supporting structures on footing or mat foundations over the clay, or on friction piles in the clay. It is recommended that tunnels, ramps, viaducts, and other structures within the area influenced by the South Bay fill be supported on piles driven through the clay to bearing on glacial till or bedrock. For preliminary design, 120-ton total capacity is recommended for the end-bearing piles. Suitable pile types include steel H, precast concrete and concrete-filled steel pipe piles.

Long-term settlement of the clay stratum will create significant downdrag loading on piles. Drag loads on piles driven through areas of deeper clay are calculated to be approximately equivalent to the pile design capacity.

In order to reduce downdrag loads, it is recommened that bitumen coating be applied full length to the piles. A suitable coating consists of asphalt cement applied hot on site for a minimum thickness of 1/8 inch. Based on field test results, bitumen-coating is effective in reducing downdrag loads on piles to approximately ten percent or less of the total potential drag load.

Installation of piles under conditions of limited headroom is required during construction of new ramps under the existing Central Artery viaduct. Open-ended steel pipe or steel H piles may be driven or jacked providing there is a minimum vertical clearance of approximately 15 feet, to accommodate the pile hammer or jacking equipment. The piles consist of short segments welded together as installation progresses. A design capacity of 100 tons compression is recommended for these piles.

A working mat of pervious, granular material should be provided under structure foundations to facilitate drainage of the subgrade, and to reduce stresses and disturbance in the clay due to construction equipment and activity. For preliminary design, the working mat should have a thickness of 24 inches, consisting of a 6-inch thick layer of sand and gravel placed directly on the natural subgrade and overlain by an 18-inch thick layer of crushed stone.

A 2-foot thick gravel and stone working mat is considered generally suitable for support of construction equipment directly on the clay or other inorganic soil strata. It may be necessary for the contractor to provide a greater thickness of granular fill, a fabric membrane, or timber mats to provide additional support for heavy equipment such as pile driving rigs operating above the organic silt layer.

Pile foundations are also required to support the tunnels over the Red Line subway tunnels at both crossing points in the Fort Point Channel, and to support the South Boston tunnel in the area between the Red Line tunnels and the easterly shore of the channel. Direct soil support at the Red Line crossings is not feasible because of potential damage which may occur to the existing tunnels due to the increase in soil stress.

Based on preliminary design

considerations, low-displacement type, steel H piles appear to be the most suitable pile for support of the structure over the Red Line tunnels. Driving steel H-piles will cause less ground movement and vibration compared to other pile types. However, steel H-piles are currently more expensive than precast concrete or concrete-filled pipe piles and require special provisions for corrosion protection as noted below. Selection of pile type should be based on further, final design studies.

Driving of steel H-piles may create sufficient vibration and ground displacement to influence stress conditions at the Red Line tunnels. Installation procedures should be considered during final design to further reduce effects of pile driving. Measures to be considered include pre-augering and controlled rate and sequence of pile installation.

Steel piles are susceptible to corrosion due to the organic soils, the marine environment, and stray electric currents which may be generated by the subway. The potential for stray current corrosion of the piles and other steel in the structure should be evaluated under final design. It is recommended that the preliminary design include a cathodic protection system for the pile foundations.

Soil-supported mat foundations are suitable for ramps and tunnels located more than approximately 100 ft. from the edge of the South Bay. These areas are considered beyond the influence of the settlements in the deep clay stratum that will occur due to South Bay filling. Soils at bearing level consists primarily of clays. In some areas, it will be necessary to excavate and replace limited thicknesses of organic soils that occur below subgrade level.

Pile installation may cause heave and subsequent settlement of the adjacent ground and structures. In order to reduce heave to tolerable limits, pre-excavation or pre-augering should be done in advance of driving displacement-type piles such as precast concrete or pipe, at locations adjacent to existing structures. Alternatively, "low displacement" type steel H-piles may generally be utilized without pre-excavation.

It is expected that existing wood pile foundations will be encountered throughout the area, as remnants from previous piers, bridges, and other structures. Wood piles which interfere with placement of new piles should be extracted.

Lateral Support of Excavations

Excavations for tunnels in the South Bay area will range up to 60 ft. deep on land and to a depth of up to 90 ft. below water level in the Fort Point Channel. Temporary lateral support will be required for these excavations. Suitable support methods consist of interlocking steel sheetpiling and concrete trench walls. Slurry walls are more applicable to the deeper excavations. Either internal cross-bracing or tiebacks are feasible for support system bracing.

It appears that the relatively soft soil conditions, and the presence of adjacent new and existing structures, may prevent the use of ground anchors in many areas for lateral support. Therefore, internal cross-bracing will be required in these areas. The bracing system should be designed to accommodate pile installation and the tunnel or ramp construction.

An important aspect of cofferdam design, especially in areas underlain by deep clay, is stability relative to bottom heave. As applicable to excavations greater than about 30 feet deep, additional stability can be achieved by increasing the depth of sheet pile penetration below final subgrade level. For cuts over 50 feet deep, stability may be provided by benching the cut, combined with increased sheet pile penetration. This approach may not be feasible where space is

limited. As an alternative, deep concrete slurry trench walls may be utilized. For stability, these walls should be extended into the glacial till below the clay stratum.

Where possible, concrete slurry walls should be located such that they may also serve as the permanent tunnel walls. Temporary concrete slurry walls oriented transverse to tunnels, as required due to construction staging, are considered practical. These walls may be left in place after the first stage construction is complete, to be used again to retain earth adjacent to excavation on the opposite side of the wall during subsequent construction stages.

To accomplish the staging, it appears feasible to incorporate the transverse slurry walls into the new tunnels using reinforcing bars embedded in the slurry walls. This method would, however, require demolition of those portions of concrete slurry wall within the interior limits of the tunnels. Steel sheetpiling may also be used for segments of lateral support systems transverse to tunnels. Although costs to remove the sheetpiling may be less than that to demolish portions of concrete slurry walls, it will probably be more difficult to efficiently incorporate the sheetpiling into the staged lateral support construction than to use slurry walls.

At locations where deep lateral support using slurry walls is required over water, particularly for the Central Artery Northbound tunnel in the Fort Point Channel, and where lateral support system installation is staged to be installed prior to general site filling, local embankment fills will be placed to facilitate concrete slurry wall installation. In the Fort Point Channel, parallel to the Red Line tunnels, a temporary sheet pile wall will be required to retain embankment fill so it does not encroach on the zone of influence of the Red Line.

The lateral support system for the Central Artery Northbound tunnel excavation in the Fort Point Channel must be properly designed and installed to limit movements of the granite block sea wall. The deepest portion of the Central Artery Northbound tunnel is near the South Boston tunnel crossing. Here the excavation for the Central Artery Northbound will be as much as 100 ft. below mean water level, extending up to 70 ft. below the base of the timber pile supported sea wall.

To limit deformation of the sea wall and to provide a relatively rigid and watertight lateral earth support system for this deep excavation, reinforced concrete slurry walls will be used. The deeper slurry walls may require additional stiffening by use of thicker walls or using T-shaped panels to increase moment capacity of the wall. Additional toe penetration and heavy internal bracing elements will probably also be required. The use of tiebacks will to some extent be restricted on the east side by the proximity of the Red Line tunnels and on the west side by the presence of sea wall timber piles.

Special considerations to protect the Red Line subway tunnels in the Fort Point Channel preclude concrete slurry wall construction for the lateral support of either the Central Artery Northbound tunnel or South Boston tunnel excavations at tunnel crossings. Filling in the channel to build slurry walls over the Red Line tunnels would overstress the subway tunnels and should not be done. Accordingly, interlocking steel sheetpiling installed by floating equipment, is generally applicable for the Central Artery Northbound tunnel north of Station 94 and for the South Boston tunnel east of Station 81.

Where the South Boston tunnel crosses the subway tunnels, it is planned to first build the tunnel section on one side of and above the subway tunnels, with the other half of the channel left open as an outlet for

the flow from the diverted Roxbury Canal Conduit. It is recommended that sheetpiling for cofferdams which parallels the subway tunnels at this location be located at least 10 ft. beyond the outside edge of the subway tunnels. The completed, first stage tunnel structure will provide a portion of the enclosure for the remainder of the tunnel.

Considering the soil profile and presence of the subway tunnels, it will not be possible to achieve significant toe penetration for this sheeting over the subway. It is recommended that a thin, impervious blanket be placed on the Channel bottom outside these sheeting segments to prevent high seepage gradients and instability of the excavation bottom. Nylon fabric, filled with pumped concrete, is considered suitable for this purpose.

Lateral support installation under low headroom conditions is required for ramps AT-SR and AT-CS under the Central Artery. At these locations, where excavation depths range up to 50 ft., it appears that concrete slurry walls will be most suitable. Existing Central Artery pile foundations will likely obstruct slurry wall installation at a few locations. This condition may require local changes in alignment of the wall, or the use of soldier piles and timber lagging for short lengths of wall.

Dewatering

Tunnel and ramp subgrades throughout the South Bay Area and the Third Harbor Tunnel subgrade in the Fort Point Channel are generally underlain by 40 to 80 ft. of clay. Groundwater seepage into excavations in these areas is expected to be minor, such that dewatering may consist of pumping from sumps.

Further along the Central Artery Northbound tunnel (north of Station 76), pumping from a deep well system will be required to depressurize the fractured, decomposed Argillite bedrock which underlies the clay and glacial till strata and is near the excavation bottom. In the vicinity of Station 80 to Station 85, the top of rock is approximately 5 ft. below the bottom of excavation; the depth to rock increases further upstation. Unbalanced hydrostatic pressures in relatively pervious rock may result in uplift, heave, and possibly failure of the bottom of the excavation. Deep well pumping would prevent these problems from occurring.

From Station 91 to Station 100, a sand layer underlies the clay stratum, at depths ranging up to 25 ft. below the proposed bottom of excavation. Dewatering of this sand layer will be required to prevent excessive hydrostatic uplift pressures on the clay below subgrade. It may be done by pumping from wells within the excavation.

Dewatering or depressurization of deep granular strata and/or bedrock may induce consolidation in the clay. Several inches of long-term consolidation settlement could result. Piezometric levels should be monitored during construction. Recharging and/or grouting may be required to reduce the influence zone of excavation dewatering.

Effects on Adjacent Structures

Comments regarding the effects of the South Bay filling on the existing structures to remain are summarized in the following paragraphs. Estimated settlements are preliminary and are intended to indicate the approximate magnitude of anticipated movements.

West Fourth Street Bridge and Roxbury Canal Conduit. These structures are located beyond the south end of the South Bay. Considering the in-situ stresses under the existing embankment at the west approach to the bridge and overlying the conduit, the limited thickness of new fill near this area, and the lesser thickness of clay under this end of the bay, minor, tolerable

settlements are estimated at these structures.

Settlements are estimated to be approximately one inch at the north end of the existing conduit, and significantly less under the west abutment of the bridge, which is supported on timber piles.

Central Artery. The Central Artery is located 120 feet or more to the west of the proposed fill. The Central Artery is beyond the area of significant influence of the South Bay fill and should not be affected.

Broadway Bridge, Railroad Bridge. These bridges are supported on timber friction piles driven into the clay. Bridge foundations, which are located in the South Bay or within approximately 50 feet beyond the edge of the bay, are likely to undergo excessive and damaging settlements due to the filling. Accordingly, it is recommended that the railroad bridge be reconstructed on new, bitumen-coated, end-bearing piles.

Railroad Wye Connector Bridge. This structure is being built across the South Bay. At the time of this study, however, the wye connector was not designed and, therefore, its foundation type was not known. If supported on friction piles or uncoated end-bearing piles, the filling of South Bay will result in excessive settlement or overstressing of the piles. For these cases, replacement of the wye connector on coated, end-bearing pile foundations will be required. Replacement may not be required if the bridge is founded on coated piles at locations which do not interfere with the proposed tunnel structures. The design of this bridge should be reviewed during the design phase for this project.

Adjacent Railroad Tracks and Equipment. Tracks located at the edge of the filled area will undergo long-term settlements, ranging in amount up to approximately six inches to one foot. Settlements will decrease in magnitude with increasing distance from the filled area. Some periodic releveling of tracks nearest the South Bay fill will therefore be required following construction.

The proposed ramps located beyond the northerly limits of the South Bay will cross underneath existing tracks and signal equipment. Consideration was given to tunneling the ramps beneath the tracks to avoid removal and replacement of special signal equipment. It was determined that tunneling for these ramps is not feasible and cut-and-cover construction is recommended.

Red Line Subway Tunnels. The proposed tunnel alignments will run above the existing MBTA Red Line twin tunnels near the intersection of Dorchester Avenue and Summer Street bridge in Fort Point Channel and opposite Gillette Co. in Fort Point Channel. In these areas the Red Line runs in a nearly north-south direction, whereas the proposed Central Artery northbound will cross approximately in a northeast - southwest orientation, while the Third Harbor Tunnel will cross approximately in a Northwest - Southeast orientation.

At their closest position in elevation, the clear vertical distance between the top of the Red Line tunnels and the bottom of the proposed Central Artery northbound tunnel section will be 4 feet. The clearance between the bottom of the proposed Third Harbor tunnel and the top of the Red Line tunnels will be 2 feet. Preservation of the structural integrity of the Red Line tunnels is an essential design and construction requirement.

The Red Line tunnels were constructed in 1915 and 1916 as two, parallel, single-track circular tunnels. Tunnel excavation was made by utilizing a circular shield, approximately 24 feet in diameter, under compressed air. As initial, temporary support, a 9-inch thick wood lining was installed. Subsequently, a 2-foot thick unreinforced concrete

permanent lining was cast. The inside diameter of each Red Line tunnel is 18 feet

At the two sites of the Red Line/Tunnel crossings, the Red Line tunnels were driven through a mixed face of clay and glacial till. The clay layer extends upward from approximately the spring line of the existing tunnels. The base slab of the proposed tunnels will be within and near the top of the clay stratum.

At the Red Line crossings, the proposed tunnels will span over the existing twin tunnels on end-bearing piles.

Construction of the proposed Central Artery Northbound tunnel, at the Summer Street site, will have the following sequence:

a. structurally reinforce the Red Line tunnels from within,

b. construction of an interlocking sheetpile cofferdam,

c. excavation of channel bed to average elevation -23 (note that the crown of the existing twin tunnels below is at an average elevation -34),

d. dewatering of the cofferdam area to the bottom of excavation,

e. complete excavation,

f. casting of tunnel slab and the tunnel section,

g. removal or cutting off sheeting.

The construction of the Central Artery northbound tunnel, will change the presently existing stress field in the soil around the Red Line tunnels, and thus the stress field in the two tunnel linings. In order to assess the effect of construction on the integrity of the Red Line tunnel linings, a preliminary analysis was undertaken, which included three parts as follows:

a. Estimate state of stress in the tunnel linings under present (i.e., pre-construction) conditions. This stress level is presumed to be acceptable based on the observed satisfactory condition of the tunnels.

b. Estimate state of stress in the tunnel linings for the condition of maximum stress change during the construction.

c. Compare the states of stress obtained from Part a and Part b.

Two alternate methods were utilized to estimate the stress field in the tunnel lining: approach proposed by Peck, et. al. (1972), Army Corps of Engineers (1969).

These analyses have disclosed that the induced stress distribution in the Red Line tunnel linings due to the proposed construction will be different from that for the existing, pre-construction conditions. The changes in the linings of the Red Line tunnels at the Summer Street crossing that will result from the excavation and dewatering required to construct the Central Artery northbound tunnel will necessitate strengthening of the Red Line tubes prior to excavation. Steel ribs can be installed wihhin the existing tunnels to provide the required additional support. There will be minor stress increases in the linings of the Red Line tunnels due to the retained fill required to install the slurry walls for the Central Artery northbound tunnel south of Station 94. However, the peak tensile and compressive stresses imposed on the linings due to the temporary filling are not expected to significantly exceed those estimated for the pre-construction conditions and are considered tolerable.

The Third Harbor Tunnel profile, allows only 2 ft. of clearance between the bottom of the proposed Third Harbor Tunnel structure and the underlying Red Line subway tunnels. The changes in stresses in the linings of the Red Line tunnels that will result from the excavation and dewatering required to construct

the South Boston tunnel will necessitate strengthening of the Red Line tunnels prior to sheet pile installation or excavation closer than 20 ft. Steel ribs can be installed within the existing tunnels to provide the additional support required.

In the Fort Point Channel, the top of the Third Harbor Tunnel will be 5 ft. above the channel mudline and may cause 5 ft. or more of sediment to accumulate throughout the area behind the tunnel. The Red Line tunnels in this area should also be strengthened to carry this possible increased load.

Where the Third Harbor Tunnel crosses the Red Line tunnels, a resilient cushion should be installed on the crown of the Red Line tubes to avoid development of stress concentrations which may occur if the tubes or new tunnel should undergo minor differential vertical movements.

Piles driven to support the proposed tunnels in the vicinity of the Red Line tunnels should be low displacement-type, steel H piles. It may be necessary to utilize predrilling and/or controlled sequence and rate for installation of piles closest to the tunnels in order to reduce soil displacements at the Red Line tunnels.

To reduce effects on the existing tunnels, a minimum 10 feet horizontal distance should be maintained for sheetpiling parallel to the tunnels. A minimum one foot vertical clearance should be maintained between the tunnel crowns and the tip of sheetpiling driven over the tunnel.

During construction, an instrumentation program should be implemented to monitor effects of the construction on stress and displacement in the existing tunnels. Criteria should be established for maximum tolerable changes in stresses and displacements. Modified construction procedures should be planned and implemented in the event that observations indicate that maximum stress and displacement levels may be exceeded.

In summary, provided that precautionary design and construction procedures are followed, it appears that the proposed tunnels may be constructed without adversely affecting the conditions of the existing Red Line tunnels.

Fort Point Channel Sea Walls. The Central Artery Northbound tunnel is planned to be located approximately 30 ft. from the face of the westerly Fort Point Channel sea wall. The sea wall is founded on timber piles of unknown length.

Inward deflections of the lateral earth support system along the sea wall (reinforced concrete slurry walls and steel sheet piles) will result in settlement and horizontal deformation of the sea wall. The magnitude of deformations of the wall will be related to the lengths of piles supporting the wall, the design and installation of the excavation lateral support system, dewatering effects and other details. Movements of the sea wall can be controlled to tolerable levels with proper excavation and lateral support construction procedures and with measures to limit effects of dewatering.

South Postal Annex. The South Postal Annex foundations consist of both caissons bearing on the glacial till stratum (original building) and end-bearing, 16-in. diameter concrete-filled steel pipe piles. It is believed that the structure's lowest floor slab is structurally supported. The proposed ramp and tunnel construction should not have significant effects on the South Postal Annex.

Gillette Company. The Gillette Company buildings are located 60 ft. or more from the shore of the Fort Point Channel. The building nearest the Fort Point Channel is the Z-building which is supported on pressure injected footings (Franki

piles) bearing in the glacial till, below the clay stratum. The bottom of the clay is approximately 60 ft. below ground surface. Sediment accumulation in the Fort Point Channel behind the South Boston tunnel is not expected to cause settlements at the Gillette Company Z-building. Other Gillette Company buildings are more than 300 ft. from the Fort Point Channel and are not expected to experience settlements due to sediment accumulation. Ground settlement along the edge of the Fort Point Channel should be less than 2 inches and will decrease to negligible amounts at about 100 ft. from the Channel.

Fort Point Channel

Early in the study, the feasibility of a "low profile" alignment in the Fort Point Channel which crossed under the MBTA Red Line Tunnels at Summer Street and would be constructed as a bored tunnel, was evaluated. It was considered that a bored tunnel was not feasible due to the following geotechnical problems:

1. Ground settlement above the tunnels may cause significant damage to the MBTA Red Line tunnels and the sea wall in the Fort Point Channel.

2. Tunneling will be very difficult and expensive, due to mixed face conditions. Clay, sand, glacial till and bedrock all will be encountered within the depth of the tunnel.

3. Tunneling will interfere with or cause settlement of foundations of existing bridges.

4. Cofferdams on the order of 100 feet deep or greater, are required at the inner end of Fort Point Channel for transition to cut and cover.

The plans as developed are for a "high profile" tunnel which crosses above the Red Line tunnels at Summer Street and carries relocated Dorchester Avenue above the tunnel. The tunnel is to be constructed by the cut-and-cover method.

Decking Between Tunnel and Dorchester Avenue

It is recommended that decking for cover, or the Relocated Dorchester Avenue spanning between the tunnel structure and existing Dorchester Avenue, be structurally supported. Preliminary analysis indicates that the pile-supported sea wall is capable of supporting the decking.

Intake Pipe for Gillette Company

A temporary intake pipe is proposed during construction as a possible mitigating measure along the southern side of the Fort Point Channel, from the Gillette Company plant to a point several hundred feet north of the proposed Third Harbor Tunnel. It appears that the soil at pipe invert level is primarily soft, organic silt, which is unsuitable for support of the pipeline. It is recommended that the intake pipe be supported on end-bearing piles, driven through organic silt and clay to glacial till or bedrock.

5.5.3 Central Area (Congress Street to Causeway Street)

General

Depression of the Central Artery will involve cut and cover construction of a below-grade, rectangular-shaped box. The mainline structure for through traffic will typically be approximately 150 ft. in width with a height up to 42 ft. The structure will accommodate four to six traffic lanes each way. The mainline box will be buried below finished grade from approximately Northern Avenue to Causeway Street. Cover will vary from about 2 ft. at State Street to about 20 ft. near the Sumner and Callahan Tunnels. Several entrance and exit ramps will be provided. Near Northern Avenue, one-way traffic connections will be made to the Dewey Square Tunnel (southbound) and a new northbound tunnel connecting from the Fort Point Channel. Near Causeway Street, connections will be made to the North Area which involve depressed

box, depressed boat, and viaduct structures.

Throughout construction of the depressed section, through traffic must be maintained on the existing Central Artery elevated structure. (Construction staging will of necessity result in traffic interruptions on the surface roadway system). Excavations for the depressed section will extend laterally below either a portion of the width or the entire width of the elevated structure, depending on station location. Excavation depths up to 80 ft. will be required.

Construction Method

Basic Requirements

In order to provide for uninterrupted through traffic, it is essential that the existing elevated structure be securely supported throughout the excavation and subsequent construction period. In addition, a substantial lateral support system will be required to retain the earth and to limit ground movements adjacent to the excavation.

Existing Central Artery Foundations

Bents for the elevated structure are supported on spread footings from approximately Broad to High Streets. Accordingly, the structure must be underpinned in this area to permit excavation. From Broad Street to beyond Causeway Street, the structure is supported on piles. The piles are steel H piles and cast-in-place concrete piles. Available data indicate that all piles within individual caps are of the same type. Many pile caps contain both vertical piles and inclined piles. Most inclined piles are battered in a direction transverse to the alignment of the existing Central Artery. An evaluation was made during this study as to whether the existing piles could continue to be used to support the Artery structure as excavation proceeded around them for the depressed section. Data available during the time for this study indicate that the H-piles were intended to be driven to rock or practical refusal and the cast-in-place concrete piles were to be driven to firm soils. Although driving records for individual piles are reported to exist, they were not readily available for review.

The existing elevated structure does not exhibit evidence of foundation distress based on available information. However, important factors relative to the in-place conditions of the piles are unknown as they influence the construction method for the Artery depression. For example, the degree of corrosion, and hence structural deterioration, which might have occurred to the H-piles is unknown. It is possible that corrosion has occurred where they are embedded in miscellaneous fill and organic soils. Additionally, stray current corrosion might have developed at the H-piles because of direct currents from adjacent MBTA facilities. The same can be said for steel casings for the cast-in-place piles, if the casings were designed to contribute to structural capacity. If significant corrosion were disclosed as excavation proceeded, costly delays might occur as remedial measures were implemented.

Those piles driven to bearing within soil can experience a reduction in bearing capacity as the excavation proceeds. This is due to decreases in overburden stresses at the pile tip as excavation proceeds. Such effects are potentially very significant where excavations of tens of feet in depth are involved. Additionally, at some locations, excavations will probably extend below the pile tips.

There are considerable field data throughout Boston to indicate that significant shaft friction can develop on pile shafts following driving. Many piles in the Boston area are designed as friction piles. Friction has also been found to develop on the shafts of piles

designed in end bearing, with this friction contributing to the pile capacity during test loading. The existing Artery piles have been in place for many years and significant friction has undoubtedly developed on the shafts. The specific proportion of present shaft resistance and tip resistance for the piles is not known.

As excavation proceeds, shaft friction, and hence the frictional component of the resistance to applied loading, will of necessity decrease as soil is removed from around the pile shafts. This will result in increased loading being transmitted to the pile tips. At the same time, pile tip resistance can drop because of reductions in overburden stress caused by the excavation. The combined result will be a redistribution of stresses within the piles as excavation proceeds and a tendency for reduction in bearing capacity at the tips of piles bearing in soil and significantly weathered bedrock. Pile behavior under these circumstances can not be predicted with confidence as it relates to secure support of the elevated structure. This is particularly true in those areas where very deep excavations occur and/or where excavations extend to near the pile tip elevations.

On the basis of the above theoretical considerations and uncertainties concerning the present in-place physical conditions of the piles, it has been concluded for purposes of this study, that the piles should not be used for continued support of the Artery during excavation. Another means of supporting the elevated structure must be devised to serve during the excavation for and construction of the depressed structure.

It should be noted that detailed evaluations in project design might indicate that it is technically feasible to use the piles in selected locations, for example, in areas where the required excavation is relatively shallow and pile tips are well below the bottom of the excavation grade.

Selected Scheme

An independent support system for the elevated structure consisting solely of new piles and/or caissons could be provided. However, such a system would be wasteful in terms of cost and time. This is so since in effect such a system would involve construction of replacement foundations, for nearly all of the elevated structure, which would only have useful life spans of a few years. Consideration was therefore given to a support scheme which could be incorporated partially within the excavation lateral support system.

There are numerous factors discussed below which influence the choice of the excavation lateral support system. Systems considered include reinforced concrete diaphragm walls installed by the slurry trench method (slurry walls), steel sheet piling, and soldier piles and lagging. Each of these systems would require heavy internal bracing or closely-spaced ground anchors (tiebacks) to resist the large earth and water pressures expected to be imposed on the walls along the project alignment.

Low Overhead Room

Because of the proposed alignment, much of the lateral support system must be installed below the existing elevated structure. Slurry walls, steel sheet piling, and soldier piles and lagging can all be installed within the overhead room typically available. However, steel sheeting and soldier piles would have to be installed in short segments, with welding used at splices. Steel sheet piling suffers a particular disadvantage because numerous splices would be required for each sheet, and since the sheet piles would be continuous longitudinally, a great number of splices would be required.

Existing Piles for Elevated Structure

The existing battered piles

pose a serious interference problem. For practical purposes, it has been determined that neither the slurry wall nor sheeting schemes can penetrate through the piles. Soldier piles could, however, be installed between the existing battered piles. Positioning the slurry trenches or sheet piling outside the interference limits of the piles appears to be the only practical solution for these two systems.

Buried Wharves, Seawalls, Foundations and Other Features

Buried timbers, wood piles, stone blocks and concrete, and abandoned utilities pose obstructions to all lateral support system installations. The soldier pile and lagging scheme is probably impacted least since the piles can sometimes be repositioned to avoid interferences. Pre-excavation of obstructions is necessary for sheet piling and very desirable for slurry trenches. Generally, except for reinforced concrete and steel, slurry trench excavations can be extended through the anticipated obstructions, although very expensive and time consuming delays may result.

Adjacent Structures

There are many buildings and utilities adjacent to the alignment which will require support during excavation. The more rigid the lateral support system, the smaller the expected horizontal and vertical movements of the buildings during excavation. Slurry walls can be designed to be more rigid than both the steel sheeting and soldier piles and lagging systems and hence are preferred.

Groundwater Cutoff

The excavation will proceed many feet below the equilibrium groundwater level. It will be important to limit groundwater drawdown outside the excavation to minimize consolidation settlements of the organic and silty clay soils present along the alignment, as well as to protect wood piles which support many adjacent buildings.

The slurry wall procedure is the preferred lateral support system with respect to groundwater cutoff. Experience has shown that slurry walls can be very effective in groundwater cutoff. The steel sheet piling wall is the next most desirable of the systems considered, although not as effective because of potential interlock leakage. Potential also exists for discontinuities in the wall because of ripped interlocks and distorted sheets due to driving difficulties. Soldier pile and lagging walls are not effective as groundwater cutoff because of leakage through the lagging.

Vertical Support for Central Artery Structure

The slurry wall can be made three ft. or more in width. The wall, therefore, can support substantial imposed loads if extended to suitable bearing materials such as glacial till or rock. Consequently, the wall can be incorporated in the underpinning system for the elevated Artery. By comparison, conventional construction of sheet pile and soldier pile and lagging walls does not result in capacity large enough in end-bearing to support imposed loads such as those from the Central Artery.

For the reasons described, slurry walls have been selected as the principal lateral support system element for the Artery depression for several reasons.

- The walls can be installed within the low overhead room present below the existing elevated Artery.
- Most obstructions, with the exception of steel or concrete piles and reinforced concrete can be removed from within the trenches where encountered, although very expensive delays or procedures may be required.
- With proper bracing the walls

will provide secure support for the elevated structure and will be relatively rigid with respect to limiting movements of adjacent soils.

o Continuous walls will serve effectively as groundwater cutoff in soil.

o The walls will have sufficient vertical capacity in end bearing to permit them to be incorporated within the support system for the elevated structure.

o The slurry walls may become part of the final structure.

It should be noted that lateral support systems other than slurry walls can generally be used where underpinning of the existing elevated structure is not required during excavation, provided the rigidity of the slurry wall is not required for limiting movements of adjacent structures. This is the case for selected ramp locations and some areas within the Fort Point Channel. Soldier piles and lagging are suitable on land where groundwater drawdown effects from dewatering or seepage into the excavation are not of concern.

Regardless of the type of lateral support system, the sidewalls must be restrained laterally. This is normally done either by a system of interior bracing (cross-lot struts) or tiebacks (drilled-in ground anchors).

In general, tiebacks are not considered applicable for restraining excavation side walls, either in excavations below the elevated structure or elsewhere. While tiebacks may be desirable or required at selected locations, cross bracing will, in general, be used. This is so for several reasons:

o Tiebacks could cause interferences to facilities which may be built adjacent to the alignment following Artery depression.

o The proposed depression passes adjacent to numerous private properties. Many easements would be required to facilitate general tieback use, and legal delays might be experienced.

o Many pile supported structures exist adjacent to the alignment. The piles could cause interferences to tieback installation.

o Certain soil conditions are not suitable for tiebacks, for example, the thick deposits of fill and organic soils near Causeway Street. Potentially, very long tiebacks would be required in these areas to extend to the deeper glacial till and rock.

o Tieback use is restricted because of certain adjacent structures. For example, at the Orange Line, tiebacks could only be used if they extended below the tunnel. Certain construction adjacent to the alignment is also now proposed. If built prior to Artery depression, it is anticipated that some of this construction might restrict tieback use in a manner similar to the Orange Line.

In summary, the slurry wall method is the scheme selected for use in partially supporting the existing elevated structure and for restraining the soils during excavation. Internal cross bracing is the principal method chosen for restraint of the walls.

Principal Geotechnical Features of Selected Scheme

Planning for this study has assumed that the slurry walls will need to extend below the lowest excavation level and into glacial till, starting from approximately Station 114, and extending continuously to Causeway Street. In project design, studies should be made within the areas of greatest depth to glacial till to determine the technical feasibility and possible cost savings associated with using bearing pile support for the walls and terminating the walls below the excavation level but above the till.

For support of the elevated Artery, pairs of cast-in-place, reinforced concrete grade beams will be constructed at each bent to span between the slurry walls, one beam on each side of the bent column. Provisions will be made to posttension the grade beams before excavation proceeds. The grade beams will be connected to the walls in such a manner as to provide rigid lateral support to the walls as well as to transfer some of the weight of the elevated structure into the walls. Pile caps for the elevated structure will be supported by transversely oriented needle beams which in turn connect to longitudinally oriented transfer beams connecting to the grade beams. Supplemental cross braces will be installed near ground surface between bent locations as necessary to provide adequate lateral support and rigidity to the tops of the walls.

All grade beams spanning between slurry walls will typically be supported by two reinforced concrete slurry piles at interior locations. These slurry piles will consist of reinforced concrete columns installed by slurry trench methods in a manner similar to the walls. They will be extended into the glacial till a sufficient distance below the excavation level to achieve the required capacity in end bearing. The piles will have sufficient plan area and column stiffness such that they do not require lateral bracing as the excavation proceeds. That portion of the elevated structure loading (plus other loads such as from surface decking) not supported by the slurry walls will be resisted by the slurry piles.

Slurry piles have been selected, for the purposes of this study, as the most feasible method of providing the required interior supports. Drilled-in caissons have been judged impractical because of the probable difficulties associated with installation in low head room. Pile foundations are also considered unfeasible for reasons similar to those discussed previously in connection with expected behavior of the existing Central Artery foundations during excavation. More economical solutions may be identified during project design.

As excavation is carried out below the grade beams, steel cross bracing will be installed and post tensioned. For the purposes of this study, it has been assumed that continuous steel wales will be required to effectively transfer bracing loads into the slurry walls. Multiple levels of steel cross bracing and wales will be used. Each cross brace will be supported at one or more interior positions to reduce the span length. Additional slurry piles will be used for this support as well as for supplemental support of the surface decking.

The geotechnical features of the selected construction scheme will provide for rigid lateral support of the slurry walls and elevated structure. The slurry walls will be capable of resisting all imposed lateral forces from the elevated structure. Additionally, the slurry walls will act monolithically in a longitudinal direction to distribute imposed vertical loads. Thus settlement of the elevated structure will be minimized. Provisions can be provided for minor releveling of the elevated structure, should that be necessary.

Other Aspects of Lateral Support Systems

Location plans of the slurry walls in the Central Area are shown in Figures 51 and 52.

As shown, the slurry walls are jogged or extended outward from the main wall alignments at several locations. This has been done to reduce the potential for interferences to the wall excavation by the transversely battered piles which support the elevated Artery structure. As discussed previously, there are also numerous historical subsurface features along the

alignment which are potential interferences to slurry wall construction. Therefore, in project design it may be desirable to modify the wall alignments.

There will be instances where the battered piles can not be avoided, for example, where the slurry wall passes below the elevated structure and between outer and center pile caps both having battered piles. At interference locations, special procedures will be required to complete the excavation sidewall. Approaches considered feasible include:

- Excavate the slurry wall to the maximum depth possible. Prior to concreting the wall, drive vertical H-piles between the battered piles in positions such that they can subsequently be used as soldier piles for a lagged wall. Excavate as normal during the excavation depth for which the slurry wall could be cast. Use soldier pile and lagging procedures below the bottom of the cast slurry wall, with appropriate groundwater control, as required.

- Use soldier piles and lagging full depth, with appropriate groundwater control as required.

- Use soil freezing and a cast-in-place reinforced concrete excavation side wall, cast from the top downward, where soil and adjacent structure conditions are appropriate.

Slurry wall thicknesses of three and four ft. are planned. The thicker wall will be used in the deeper excavation areas. In project design use of four ft. thick walls might be extended to other locations, for example to limit applied bearing pressures at the bases of the walls or to reduce the amount of cross bracing required for excavation lateral support. In addition, consideration could be given to using T-shaped panels to increase moment capacity of the slurry wall.

Foundations for Structures

A mat foundation bearing on natural inorganic soil is recommended for the main box structure and all ramps in the central area. Subsurface data indicate that the structure will bear on clay or glacial till along most of the alignment. Between Stations 145 to 155, glacio-marine deposits will be present. Rock could be present at or above foundation bearing level in the vicinity of Station 150.

A compacted granular fill working mat should be provided directly below the foundation mats, as discussed in the original DEIS/DEIR, Supportive Engineering Report.

Organic soils will be present below foundation bearing level beneath the main box near Causeway Street. Fill and organics may remain under the upper part of some ramp excavations. These soils are unsuitable foundation materials and must be removed and replaced with compacted granular fill.

Vent buildings 1, 3, and 4 may be supported by mat foundations bearing on natural inorganic soil. It is expected that slurry walls used for lateral support of the required deep excavations at buildings 3 and 4 can also be used to carry a portion of the building loads. However, it is recommended that pilasters be designed to transfer the weight of the above-grade walls to the base mats in order that excessively high bearing pressures do not develop on the bottoms of the exterior slurry walls.

Between approximately Stations 125 to 145 and Stations 151 to 161 the depressed structure will be underlain by clay. Excavation for and construction of the depressed structure will result in a net stress reduction within these soils. While swelling or rebound of the clay will occur, it is not estimated to be of significance to the completed construction.

Effects on Existing Structures

Blue Line Subway Tunnel

The proposed depressed Central

Artery Structure will cross above the existing MBTA Blue Line Subway tunnel at State Street, immediately west of the Aquarium Subway Station. The Blue Line runs directly under State Street and is essentially perpendicular to the Central Artery alignment. There is approximately 32 ft. of soil above the subway tunnel at this location.

This section of the Blue Line was constructed in 1902 and 1903. The tunnel is a single, arched opening with inside dimensions about 24 ft. wide and 21 ft. high. The invert is 24 inches thick, the arch is 31 inches thick and the sidewalls are 31 to 33 in. thick, all of unreinforced concrete. The tunnel was mined through fill and clay, using multiple drifts and a roof shield, under compressed air.

Consideration was given to a "high" profile over the MBTA Blue Line tunnel and a "low" profile beneath. The Blue Line tunnel is underlain by silty clays which are in turn, underlain by glacial till. Tunneling within these soils below the tunnel poses substantial risk of detrimental settlement of the tunnel structure. The problem is particularly severe given the required width of the Central Artery structure and the need for multiple tunneling drifts. Therefore, as with previous studies for the Blue Line in East Boston and the Red Line at the Fort Point Channel, it was determined that a "high" profile over the Blue Line is required, with a pile-supported structure spanning across the existing tunnel.

A preliminary evaluation of the effects of the proposed excavation on the Blue Line concluded that changes in stress and deflection may be controlled within tolerable limits, and measures to strengthen the subway lining do not appear warranted. By comparison with the Red Line crossings in the Fort Point Channel, the Blue Line tunnel at the Artery crossing is at a higher elevation. Also, the excavation will be relatively broad in extent, and will be on land.

Considering these factors, it appears feasible to construct the Artery crossing using an intricately staged and monitored excavation process to reduce stresses and displacements in the tunnel. Further detailed study will be required during project design.

The following special construction procedures relative to crossing the Blue Line tunnel with the proposed depressed Central Artery structure are recommended for preliminary planning:

- Earth supported stockpiling of construction materials or storage of construction equipment should not be allowed within 50 feet of the Blue Line tube.

- Slurry walls which extend below the tube crown should end at least 10 feet from each side of the tube.

- A lateral support system of soldier piles and lagging or steel sheetpiling should be installed between the ends of the slurry walls and over the tube.

- Groundwater should be drawn down to or slightly below the crown of the tube during construction.
- Soil within 50 ft. of the tube should be excavated in approximately even lifts to minimize development of uneven stresses on the tube. Excavation should proceed until the crown of the tube is exposed over the width of the box.

- Piles driven to support the Central Artery box over the tube should be no closer than ten ft. from the tube. Low displacement H-piles should be used to at least 25 ft. from the tube.

- The bottom of the box will be within approximately two ft. above the tube crown. Therefore, a resilient cushion should be installed on the crown to avoid development of stress concentrations if the tube and box should undergo minor differential vertical movements.

o Clean pervious backfill should be placed below and at the sides of the box to allow free movement of water above the tube. Minimum thicknesses should be 1.5 ft. below the box and 3 ft. on the sides of the box.

o Steel sheetpiling, if used to support the excavation at the box, should be withdrawn following backfilling to allow free movement of groundwater.

In project design, consideration should be given to stray current corrosion effects on the H-piles used for support of the Artery structure over the tube. Excavation for and construction of the depressed structure will result in a net stress reduction within the clays below the tube. Preliminary estimates indicate that swelling or rebound of the clay should be less than two inches. Movements of this magnitude are not expected to be detrimental to the tube.

Orange Line Tunnel

Between Stations 153 to 154, the west side wall of the Central Artery structure will be close alongside the existing MBTA Orange Line subway tunnel.

This portion of the Orange Line was constructed in 1965. The tunnel is a reinforced concrete box, with a center wall. The dimensions of the Orange Line box vary. Nominal inside dimensions are approximately 17 ft. high by 30 ft. wide. Roof, invert and sidewalls are typically two to three ft. thick. There is approximately five ft. of soil cover above the tunnel.

The tunnel was constructed by cut-and-cover methods and is soil-supported on clay. Organic soils, where present below the invert slab, were removed and replaced by compacted gravel or concrete. In this section, the Orange Line is merging with another, older cut-and-cover tunnel on the west side, away from the Central Artery alignment. This older tunnel has a wood pile-supported invert slab.

It is difficult to maintain stability of a thin prism of soil between a slurry-filled trench and a rigid structure. Therefore, the slurry wall for the west side of the Central Artery excavation should be installed tangent to and parallel with the tunnel until there is room to step the slurry wall away at least 10 ft. from the tunnel.

Slurry wall panels adjacent to the Orange Line must be excavated in short sections, with more than the normal staggering between panels in the excavation sequence. This will help limit the development of unbalanced lateral pressures on the Orange Line structures during excavation of the slurry wall panels.

Structural analyses of the Orange Line structure will be required during project design to assess the effects of lateral loads on the tunnel from cross-lot bracing within the Central Artery excavation. It may be necessary to use a system of rakers, or tiebacks drilled in beneath, to prevent undesirable loads on the tunnel. Studies will also be required relative to the effects that general box excavation may have on movements of the subway structure.

Adjacent Buildings

Excavations required for construction of the depressed Artery will result in ground movements that translate into settlement of the ground adjacent to the excavations. These ground movements may have impacts on adjacent buildings and utilities. Depending on the potential for movement, some form of protection to limit distress of structures may be required. Possible means of protection include underpinning and compaction grouting.

The amount of expected ground movements adjacent to the depressed Artery excavations will depend on

several factors, including primarily subsurface conditions, the stiffness of the lateral support system, method and degree of difficulty of wall installation, and workmanship. It should be noted that the slurry walls will be rigidly restrained at the top by the transverse girder beams used for support of the elevated structure. Additionally, the potential for significant movements of the bottoms of the walls is considered minimal because the walls will extend into glacial till. These factors will help to limit development of ground surface movements during general excavation within the slurry walls.

To assess the potential impact of the proposed construction on adjacent structures, preliminary analyses of vertical ground movements were conducted, utilizing published and other available lateral support performance data and trends.

The lateral limits within which significant ground surface settlement would be likely to occur vary greatly, depending on the depth of excavation and subsurface conditions. Structures located close to the alignment have the greatest potential for significant movement. Based on the preliminary analyses, it appears that the structures bordering the alignment on the easterly side from Atlantic Avenue to Causeway Street will potentially be influenced most.

Further studies will be required in project design to establish the actual limits of expected significant movement for individual structures, and the extent to which structures should be protected. These studies should include relatively extensive investigation of site specific subsurface conditions, details of existing foundations, and type of construction.

Building Demolition and Underpinning

A number of existing buildings conflict with the location of the proposed structure. Preliminary studies were undertaken to assess the technical feasibility of underpinning these buildings, as summarized below:

Harbor Plaza

The Central Artery northbound tunnel will run through the Harbor Plaza building. In profile, the top of the tunnel runs slightly below the building's basement level. Studies were undertaken to evaluate the feasibility of constructing the tunnel under the building, as an alternative to demolishing the building and constructing the tunnel by normal cut-and-cover methods.

The Harbor Plaza building is bounded by Fort Point Channel and Atlantic Avenue along its southern and northern fronts, respectively. Its base area has the shape of a trapezoid with the two parallel sides (about 240-feet and 290-feet long respectively) running approximately in a northwest-southeast orientation. The average width of the 12-story building is 100 feet.

The Harbor Plaza building is supported by belled caissons. The average shaft and bell diameters for the caissons are 4 feet and 12 feet, respectively. An approximately 15-foot high crawl space exists between the first floor level and ground surface.

The subsoil formation at the Harbor Plaza site consists of the following units (from the ground surface down):

Miscellaneous Fill, including remnants of the channel seawall (granite blocks, deteriorated wood piles, debris, etc.);

Clay, with a very stiff to hard crust. The caissons supporting the Harbor Plaza building are belled out in the clay crust formation. The clay layer is about 20 feet in thickness;

Glacial Till; and

Bedrock.

Water level at the site is believed to be controlled by the tidal levels in the harbor.

The study has concluded that it is technically feasible to construct the Central Artery northbound tunnel under the Harbor Plaza Building. The method requires underpinning of nearly half of the present caissons which interfere with or are in close proximity to the proposed tunnel. Upon construction of the tunnel, the load carried by the underpinned caissons will be transferred onto the tunnel roof. The tunnel structure will be soil-supported on a mat foundation bearing on glacial till.

The underpinning procedure will include the following steps:

1. Drive interlocking steel sheetpiling around the Harbor Plaza building to control tidal water and to cut-off groundwater inflow.

2. Install sumps and dewater the fill stratum within the sheet pile cofferdam.

3. Excavate fill materials.

4. Install piles to end bearing in glacial till around each caisson to be underpinned (approximately 6, 100-ton capacity piles per caisson). Pile installation method to consist of jacking open-end steel pipe piles in short segments. After the pipes are jacked, clean inside of pipes, and place required reinforcement and concrete.

5. Roughen the concrete along caisson shaft to prepare surface for bonding the collar (ring) beam to be poured against the shaft. The collar beam will serve as a pile cap for the jacked piles, transferring load from the caisson shaft to the piles.

6. Jack preload into piles and cast reinforced concrete collar beam.

7. Install soldier piles to below tunnel subgrade on the two sides of the proposed tunnel alignment. At locations adjacent to soldier piles install wells into the glacial till for predrainage of the tunnel excavation.

8. Excavate the clay around and beneath the underpinned caissons, including bells. Place lagging between soldier piles and brace them as the excavation progresses.

9. Cut-off and remove the caisson shaft and bell below the collar beam. Excavate to tunnel subgrade, generally in glacial till. Where clay is present at subgrade, overexcavate the clay and replace with concrete.

10. Form and pour the base slab, walls and heavy roof slab of the tunnel section, with sleeves placed around such caisson shaft. Post-tension the roof slab in a transverse direction to increase flexural rigidity as required.

11. Using jacking as necessary, transfer the load carried by the underpinned caisson shaft (i.e., the uncut upper segment) onto the tunnel roof. Grout the annulus around the caisson shaft within the depth of the roof. The load transfer procedure should be planned to induce minimum deflection of the roof slab, and tolerable differential movements between column locations.

12. Remove caissons and underpinning within the tunnel and complete tunnel construction.

Due to the very limited working space, excavation, underpinning and related operations will have to be performed manually and by means of special, small size machinery. It is anticipated that the underpinning of the Harbor Plaza building will take about nine months, during which the building need not be evacuated.

The scheme described above illustrates a technically feasible method for underpinning the Harbor Plaza building to permit construction

of the Central Artery northbound tunnel. Other methods and details for underpinning may be developed. It is believed that the building can be underpinned without significant damage to the structure. It will not be possible to avoid causing some settlements; however, these can be limited to tolerable levels by: appropriate application of jacking and preloading during underpinning and final load transfer; the use of post-tensioned concrete construction; and other methods.

Dewatering will result in some increased vertical effective stress and consolidation in the clay stratum. This will result in some settlement, primarily of caissons which are not underpinned. The settlement, estimated to be on the order of 1/4 to 1/2 inch, is within probable tolerable limits for the structure.

Hook Lobster Company

The Northbound Central Artery also passes beneath the west wing of the Hook Lobster facility, located on the south side of Northern Avenue, at the west end of the existing bridge. The building is a one and two-story metal frame structure constructed largely on a wood pile-supported wharf over open water. The original structure was completed in about 1935. Additions were made in 1947 and 1954.

Underpinning of this structure would be quite difficult. The presence of the structure creates a major hindrance to construction of the proposed tunnel, especially with respect to removing an existing seawall beneath Hook Lobster and in controlling water flows into the proposed excavation from Fort Point Channel.

Therefore, for the purposes of this study, underpinning Hook Lobster has been judged to be impractical.

Other Buildings

The Post Office Complex, (211-213 Hanover Street) located on the north side of the Sumner Tunnel exit ramp will require underpinning along the south exterior wall to allow a proposed lowering of the surface grade. This work would be expected to be relatively routine and should not involve interrupting use of the facility. At least two months would be required to complete the underpinning.

As discussed in the previous section, extensive design investigations will be needed to evaluate the possible need for additional underpinning or other protective measures to limit distress that could develop in other buildings along the alignment as a result of ground settlement adjacent to the proposed excavation.

Groundwater

Slurry wall and sheet pile lateral support systems will be used where necessary to prevent "near surface" groundwater levels from falling below normal ranges. Since the lateral support systems will typically be constructed to glacial till or clay, the potential exists for interuption of usual groundwater flow patterns. Consideration should be given in project design to the need for providing for flow of groundwater across the excavation width. This may be necessary to prevent development of elevated water levels on one side of the excavation.

Construction of deep excavations requires that hydrostatic pressure within relatively pervious soil strata below the excavation subgrades be controlled as necessary to maintain bottom stability of the excavations. Excavations for the structure will frequently terminate in glacial till. The relatively infrequent pervious lenses or layers within the till are not generally anticipated to require dewatering of the till for this purpose. This is, however, a matter requiring further evaluation in project design.

The Cambridge Argillite bedrock

along the alignment is anticipated to be highly weathered near its upper surface. The degree of weathering is believed to be quite variable depending on location. While the argillite will weather into a clay, many joints and discontinuities are also expected to be present. The upper portion of the argillite, therefore, has the potential for being permeable with respect to the rate of groundwater flow and requirements for pressure relief during construction.

From approximately Station 140 to Causeway Street excavations for the main box, some of the deeper ramps, and Vent Building 4 will extend to depths where relatively thin soil deposits are left in place over the rock. Excavations into the rock may occur locally. Because of the possible permeability of the weathered rock, potential exists for uplift failure of the unexcavated soils due to hydrostatic pressures within the underlying rock.

For planning purposes it is assumed that in some locations wells will be required to be drilled into the rock and pumped in order to reduce water pressures to tolerable levels. The rates of required pumping can not be estimated with confidence based on available boring data in the project area. Several deep excavations were made to rock in downtown Boston during the last two to three decades, principally for rock caissons. Experiences from these projects are varied and similarly do not provide a basis upon which to make reliable estimates of pumping requirements, although out-of-the-ordinary equipment and procedures would not be expected to be necessary. Likewise, it is not possible at this time to predict the lateral limits to which reductions in piezometric levels will extend within the rock outside the construction limits.

Lowered piezometric levels in the rock at points outside the construction area might cause consolidation settlements of the overlying silty clay deposits, particularly if dewatering extends over many months. Whether or not the settlements would be of significance to structures is dependent on the magnitude of the settlement, the uniformity of the settlement, and foundation type. Preliminary calculations indicate that settlements may be of significance to some structures.

Therefore, for purposes of this study, it has been assumed that actions will be required to limit the magnitude and lateral extent of pressure relief within the rock outside the construction area. These actions may include grouting of the rock and groundwater recharge. Monitoring of this aspect of the construction will require using observation wells, piezometers, deep settlement points and level surveys at ground surface.

The degree of weathering of the argillite bedrock and the requirements for and effects of pressure relief within the rock are matters requiring study in project design.

The slurry walls used for underpinning of the elevated Artery will extend upward to near present ground surface and hence above normal groundwater levels. Provisions are required to prevent the slurry walls from acting as barriers to groundwater flow following construction. Near the completion of backfilling, the walls should be demolished to approximately elevation 0 at intervals of about 100 ft. Pervious soils relatively free of fines will be used to backfill these "windows" as well as to connect opposite windows over the box. This will provide for cross flow of groundwater. Where the top of the box is above elevation 0, pipes should be provided through the box structure as necessary to carry groundwater flow.

Utilities

There are a myriad of utilities which cross the alignment within the central area of the Central Artery. Early relocation of utilities into

corridors crossing the alignment will be required to permit efficient installation of lateral support systems later. Multiple utility relocations are anticipated. During utility relocations and later construction, all abandoned utility pipes which are encountered and all pipelines that are to be permanently removed from service should be securely plugged.

Rerouting of utilities parallel to the excavation is also anticipated. Pile support will be necessary where organic soils are present below the inverts of utilities which are settlement sensitive. Flexible connections should be provided between pile and soil supported sections.

Near the end of construction, the slurry walls will be demolished to approximately elevation 0 at all street crossings. This will facilitate final utility placement as well as future utility construction. The work will be coordinated with demolition of the wall due to groundwater considerations.

Future Construction

Preliminary consideration has been given to the possibility of constructing buildings above the completed box structure, once the existing elevated expressway has been removed. The building height that could be supported is limited by the structural capacity of the box and soil structure interaction effects at the box foundation level. The box itself would normally be designed to support only the weight of overlying backfill and surface traffic loadings.

Light buildings could be supported on spread footing or mat foundations within the backfill above the box structure, provided that there is a basement which results in removal of a weight of backfill equal to the total building weight. In this way, there would be no net increase in the total load supported by the box. Local stress increases may however be imposed on the roof slab, depending on footing bearing pressures and the footing levels relative to the top of the roof.

For planning purposes, it may be assumed that light structures, with basements and up to three-to-five stories above-grade, may be developed over the box, provided there will be sufficient backfill to remove.

Taller structures up to ten stories might be constructed over the box if certain conditions were met. Even with a basement, structures of this height range could impose some loading on the box greater than the weight of the removed backfill. Therefore, to permit construction of these building heights, strengthening of the box structure would probably be required. Additionally, soil-structure interaction of the foundation mat with the underlying soils and slurry piles would require evaluation.

For buildings of still greater height, it is expected that special, more costly provisions would have to be incorporated in the design of the depressed section or independent building foundation support provided.

Buildings might also be located such that one portion of their width is located over the box structure and the remaining width to the side. Special design considerations will be required to account for possible differential settlements of the buildings in these instances.

Buildings can be designed so as to transmit some of the load to the slurry walls. However, these loads must be limited so that bearing pressures at the bases of the walls do not exceed those which existed during construction of the depressed section.

5.5.4 North Area

The North Area extends from the north side of Causeway Street to the Gilmore Bridge in Charlestown. This area also includes the connectors

from Leverett Circle to the mainline alignment. The mainline alignment crosses land and water (Charles River) and the Connectors encounter the Charles River and Boston Harbor. Project design must consider the B&M railroad tracks and facilities, existing Leverett Circle structures, new bridge piers in the Charles River, the existing high level bridge piers, the existing Charles River dam and locks, and procedures for filling within the Charles River and Boston Harbor.

Types of construction planned in this area include cut-and-cover tunnels, boat sections, viaduct (overhead) structures, and two new bridges across the Charles River.

Foundations for Structures

Mainline and Vent Building No. 5

The alignment and grade of the mainline structure begins to rise and separate north of Causeway Street into the northbound (NB) and southbound (SB) roadways. Construction changes from tunnel to boat section to overhead NB and SB viaducts. The above grade portions extend from approximately Station 168 to Station 207, where they join the existing I-93 overhead structure.

Organic soils will be present below foundation bearing level for all of the mainline alignment north of Causeway Street. These soils are unsuitable foundation materials. Based on the proposed profile, it is recommended that the main box structure up to Station 164+90 (existing overhead structure, bent no. 8) be supported on compacted granular fill placed after removal of the unsuitable soils. It is recommended that all structures north of Station 164+90 be supported on end bearing piles driven through the fills and organics into the glacial till or bedrock. Pile capacities on the order of 130 ton total capacity are recommended. Suitable pile types include steel H, precast prestressed concrete, and concrete-filled steel pipe piles. It is anticipated that precast-prestressed concrete piles will be the most economical pile type.

A compacted granular fill working mat should be provided below the slabs, as discussed for the Central Area.

Viaduct approach embankments, required from approximately Station 168+00 to 169+40, should be constructed of light weight fill. The existing fill should be excavated and replaced with light weight fill to a depth equal to the height of the proposed grade to minimize post-construction settlement of the embankments resulting from consolidation of the organics. Types of lightweight fill include expanded shale or a chemical foam/cement mixture.

Vent Building No. 5 may be supported by a mat foundation bearing on naturally deposited inorganic soils or compacted granular fill placed after excavation of unsuitable soils. It is expected that slurry walls used for lateral support of the excavations can be used to carry a portion of the building load, as discussed for Vent buildings 3 and 4 in the Central Area.

The northbound and southbound roadway bridges, spanning the Charles River, will be supported on new piers constructed in the Charles River. The piers for the northbound roadway will be constructed as an extension of the existing high-level bridge piers, a span of 375 ft. The southbound bridge will require a span of 400 ft. The piers will be constructed approximately parallel to the existing shoreline, skewed to the roadway, to accommodate the required ship maneuvering and bank clearance limits.

Based on the anticipated subsurface conditions, it is recommended that the new piers be supported on piles driven to end bearing in glacial till or bedrock. Pile capacities up to 130 tons may be used. Suitable pile types include steel H-piles, concrete-filled steel

pipe piles and precast prestressed concrete piles. Piles may be installed within a sheetpile cofferdam (caisson). A tremie concrete seal should be provided in the cofferdam to permit dewatering and construction of the pier. Low displacement, steel H-piles should be used within 20 ft. of the Orange Line Tunnel to minimize disturbance and vibrations.

Leverett Circle Connectors and Ramp

A mat foundation bearing on naturally deposited inorganic soil is recommended for Connector CN-S from the mainline alignment to Station 67+00 and from Stations 80+70 to 94+00. A working mat should be provided directly below the mat, as previously noted. Organic soils will be present below foundation bearing level along Connector CN-S and Ramp CN-L from approximately Stations 67+00 to 80+70 and along Connector S-CS from the mainline alignment to Station 88+00. These soils are unsuitable foundation materials. It is recommended that these soils be excavated and replaced with compacted granular fill from approximately Stations 78+00 to 80+70 along Connector CN-S and from Stations 78+00 to 80+00 along Connector S-CS. The remainder of the connector alignments and the entire length of Ramp CN-L should be supported on piles driven to end bearing in the glacial till or bedrock. Pile capacities up to 130 tons are recommended. Suitable pile types include precast-prestressed concrete, steel H, and concrete-filled steel pipe piles.

For the segments of connectors crossing over the Orange Line, it is recommended that the structures be designed to span over the subway and be supported on end bearing piles. The minimum vertical clearance between Connector CN-S and the Orange Line tunnel is approximately four ft. To minimize additional loads on the tunnel and to facilitate construction, a tremied pile-supported relieving slab should be constructed to support the connector in the vicinity of the Orange Line. Resilient compressible material should be placed between the relieving slab and the tunnel to allow for some deflection of the slab. Low displacement-type steel H-piles should be used within 20 ft. of the tunnel. Steel H-piles will cause less ground movement and vibration compared to other piles. Steel H-piles are susceptible to corrosion due to the organic soils, the marine environment, and stray electrical currents which may be generated by the subway. The potential for stray current corrosion of the piles and other steel in the structure should be evaluated in project design. A more detailed discussion of construction in the vicinity of the Orange Line is contained later in this section. In addition, design modifications to be made during Preliminary Design to minimize impacts along the Charles River's edge may relocate the ramp and connectors which would require additional soils investigations and analysis.

Excavation and Lateral Support

Mainline

Excavation and lateral support will be required for construction of the mainline structure. It is recommended that the excavation up to Station 164+90 be supported by slurry walls, similar to that used in the Central Area, primarily to provide support of the existing elevated structure. Excavations up to 50 ft. deep will be required. The slurry walls should extend into the glacial till soil to provide adequate bearing and lateral resistance at the base of the excavation and a groundwater cut-off during construction.

At approximately Station 162+70, Connector CN-S begins to diverge from the mainline. From this point, up to approximately Station 163+90, the excavation for the mainline and CN-S may be made within the same excavation, using the outermost walls for overall support. From Station 163+90 to 164+90, the excavations may be separated, but it

will be necessary to install some bracing between the two excavations, due to the limited amount of soil available for resistance between the connector and mainline. Beyond Station 164+90, the excavations may be completely separated. Underpinning for the existing elevated structure should be the same as that used in the Central Area, up to Station 164+90.

Beyond Station 164+90, the lateral support system will not be required to support the elevated structure and there are no adjacent structures that require rigid support. Slurry wall use is therefore not required. Excavations will be made below the groundwater level, and soldier pile and lagging will not be suitable. Interlocking steel sheetpiling is the most suitable lateral support type in this area. Sheetpiling should be left in place where over excavation and replacement of unsuitable soils occurs below normal foundation level.

As discussed for the Central Area, internal cross-bracing will be required.

<u>Leverett Circle Connectors and Ramp</u>

Except as noted below, interlocking steel sheetpiling will be required for lateral support of excavations and groundwater cut-off for construction of the Leverett Circle Connectors CN-S and S-CS, and Ramp CN-L.The proximity of the alignment to the Charles River and Boston Harbor prevents the use of soil anchors (tiebacks) and internal cross-bracing will be required. As with the mainline, sheetpiling should be left in place where over excavation and replacement below normal foundation level occurs.

Excavation for Connector CN-S from the mainline to approximately Station 67+00 (Vent Building 5) will be approximately 40 ft. deep and will be adjacent to the Stop & Shop Warehouse and a portion of the Charles River Building. Slurry walls will be the most suitable lateral support system for this portion. Concrete slurry walls are also required for Connector CN-S in the vicinity of Leverett Circle due to the low headroom available and the requirements to minimize groundwater lowering in the vicinity of the existing viaduct which is supported on timber friction piles. The anticipated limits of slurry walls are from Stations 91+00 to 92+50. However, design modifications to be made during the Preliminary Design Phase will require a re-evaluation of soil conditions and foundation requirements in this area.

<u>Effects on Adjacent Structures</u>

<u>MBTA Orange Line</u>

The proposed Leverett Circle Connectors CN-S and S-CS will cross the MBTA Orange Line tunnel near the Charles River. In addition, the southbound roadway bridge piers will be constructed adjacent to the Orange Line tunnel in the Charles River and the southbound viaduct will cross the tunnel in Charlestown. Construction over and adjacent to the Orange Line tunnel will require special procedures to prevent damage to, and minimize disturbance of, the tunnel.

The Orange Line extension from Haymarket Square to the Bunker Hill Community College in Charlestown was constructed in the late 1960's. The tunnel portions immediately north and south of the Charles River were constructed in a cut-and-cover excavation as a cast-in-place, reinforced concrete box approximately 34 ft. wide by 24 ft. high. The walls, roof, and floor vary in thickness from approximately 3 to 3.5 ft. At the north and south seawalls on the Charles River, transition sections approximately 70 ft. long were constructed with 4 ft. thick walls, the Charles River was constructed as a sunken tube placed in a dredged excavation. The prefabricated steel plate tube was sunk in place and the walls, roof, and slab were filled with tremie concrete. A reinforced

concrete liner was then constructed. The total thickness of walls, slab and roof is 2 to 3 feet.

The Orange Line tunnel, within the limits of the transition structures and the sunken tube is founded on glacial till and bedrock.

Leverett Circle Connectors

At the Orange Line crossing, the connectors will span over the existing tunnel, on end bearing piles. In order to minimize disturbance to the tunnel, the support piles and sheet piles for lateral support should be maintained to a minimum of 5 ft. from the most exterior parts of the existing tunnel. Within 20 ft. of the most exterior parts of the tunnel, support piles should be low-displacement type steel H-piles.

The minimum vertical clearance between the Orange Line tunnel and Connector CN-S is approximately four ft. As previously recommended, a tremied pile-supported relieving slab should be constructed to span the existing tunnel and support Connector CN-S. Connector S-CS has sufficient vertical clearance to permit construction of the pile supported roadway slab by conventional methods.

Studies have shown that special procedures will be required to accomplish the construction in a manner that will not adversely affect the Orange Line tunnel. The existing sea wall will first have to be demolished within the limits of the work. Necessary excavation will then be performed including exposing the tunnel roof. Steel sheetpiling will be installed adjacent to and over the tunnel to form a part of a cofferdam structure which can later be dewatered. This will require special design considerations. Sheeting adjacent to the tunnel will be installed at least 5 ft. from the tunnel, as noted previously, and the sheet over the tunnel kept 1 ft. above the roof by blocking. A tremie slab will be cast as one element of the water cutoff when the bearing piles for the connector are driven. The cofferdam will then be dewatered and the connector constructed. However, design modifications to be made during Preliminary Design to minimize impacts along the Charles River may relocate these connectors. A re-evaluation and analysis of the new location will be required.

Bridge Piers

The north and south bridge piers for the southbound roadway will be constructed within interlocking steel sheetpiling cofferdams. In order to minimize disturbance to the tunnel, sheetpiles should be maintained to a minimum of 5 ft. from the most exterior portions of the tunnel. Based on the present alignment, it is anticipated that the south bridge pier will be constructed entirely on the east side of the existing tunnel. For the north pier, it is anticipated that piers will be constructed on both sides of the tunnel and that the piers will be connected by a transfer beam to support Connector CN-S.

Within 20 ft. of the most exterior portion of the tunnel, the support piles should consist of low-displacement steel H-piles. The potential for stray current corrosion of the piles should be evaluated in project design.

Mainline Crossing

The elevated southbound roadway crosses the Orange Line tunnel on a skew in Charlestown from approximately Stations 181+50 to 186+00. End bearing piles are required for support in this area. In order to minimize disturbance to the Orange Line tunnel, no piles should be installed within five ft. of the outermost part of the tunnel. Within 20 ft. of the outermost part of the tunnel, steel H piles should be used. Cost and corrosion aspects of steel H piles were discussed previously. Installation procedures should be studied during project design to

further reduce effects of pile driving. Considerations include pre-excavating and driving sequence.

The sequence of construction in the at-grade portions of the Orange Line and in the vicinity of the Community College Station should be evaluated during project design to minimize disruption to service.

B&M Railroad Crossing

The proposed Leverett Circle connectors and ramp will run under the B&M railroad lines in the area between North Station and the bridge over the Charles River. The connectors and ramp cross the rail lines on a skew, just south of the point where the lines begin to merge to cross the Charles River. It appears that a portion of the tracks in this area are constructed over a timber pile supported deck and seawall. It is understood that ten rail lines leaving North Station must be maintained in operation during construction.

It appears that cut and cover type construction will be the most suitable way to cross the B&M lines. It is anticipated that by constructing seven new temporary rail lines and maintaining three of the existing lines, one half of the crossing can be constructed. Upon completion of one half of the crossing, the rail service can be shifted back to the original track alignment, permitting construction of the other half of the tunnel crossing. Some localized temporary additional pile support or underpinning of the existing rails in the immediate vicinity of the tracks will be required during construction. Design modifications to be made during subsequent phases of the project suggest that these requirements may be revised.

Leverett Circle Structures

Construction of Connector CN-S will require excavating adjacent to and below the existing viaduct structure at Leverett Circle. The structure is supported on timber piles driven into the sand and clay underlyingthe area. The maximum depth of excavation is approximately 22 feet (approx. 16 feet below the groundwater level). Because of the low overhead below the viaduct and the need to maintain the groundwater level for the existing timber piles and to limit lateral movements, a slurry wall is the most suitable temporary earth support method at the viaduct. It is anticipated that slurry walls will be required from approximately Stations 91+00 to 92+50. Beyond these limits, steel sheetpiling can be used for temporary earth support. Design modifications to be made during subsequent phases of the project suggest that these requirements may be revised.

Spaulding Rehabilitation Hospital

The Leverett Circle connectors will be constructed adjacent to the new extension of the Spaulding Rehabilitation Hospital. The extension is founded on concrete filled steel pipe piles driven to end bearing at about elevation -50 to -60, in the glacial till and bedrock underlying the structure. It is anticipated that construction of the connectors and ramp will not have adverse effects on the facility. However, further evaluation should be made during project design to determine if slurry walls would be required in this area to minimize lateral movement of the soil. Design modifications to be made during subsequent phases of the project suggest that these requirements may be revised.

Subsurface Obstructions and Utilities

It is anticipated that construction will encounter existing seawalls, piers, wharves and pile supported decks. The plan limits of these structures should be further evaluated during project design.

Several combined sewers cross the Leverett Circle connectors and

ramp in the vicinity of Station 88+00 (CN-S) and at Leverett Circle. If fill and organic soils exist below the invert of these and other utilities which may be affected by the Connectors, it is anticipated that pile support will be required. Flexible connections to accommodate differential movement should be provided at locations where utilities cross the alignment. Design modifications to be made during subsequent phases of the project suggest that these requirements may be revised.

Construction Procedures Within the Charles River and Boston Harbor

Portions of the Leverett Circle connectors and ramp are located beyond the existing shoreline of the Charles River and Boston Harbor. Construction will require placing both temporary and permanent fill in the river and harbor. The amount of fill placed should be limited to that necessary to provide adequate stability during construction and adequate cover for the permanent structure.

Permanent fill within the river is required to provide cover for connector CN-S and ramp CN-L. The shoreline needs to be extended approximately 60 ft. between Stations 73+00 and 75+00; 120 ft. between Stations 78+00 and 83+00; and 75 ft. between Stations 83+00 and 86+50. Temporary filling to provide stability for excavation lateral support systems (equivalent lateral earth pressures on each side of excavation) should extend approximately 20 to 40 ft. beyond these limits. Temporary filling will probably also be required outboard of the existing seawall in the vicinity of Station 77.

Permanent filling in Boston Harbor to provide cover for Connector CN-S will extend the existing shoreline a maximum of 120 ft. between Stations 67+00 and 70+00. Temporary filling will extend approximately 60 ft. beyond this.

It is anticipated that the outboard sheetpiling at the connectors and ramp may remain in place to provide for permanent bulkheads in areas requiring permanent filling. In areas requiring nominal permanent fill, the existing river bank may be extended. In other areas of construction, the original river bank should be restored.

It is anticipated that cantilevered interlocking steel sheeting installed near the toes of the fills will be required to minimize disturbance to the river bottom and prohibit turbidity in the water. In addition, fill materials should consist of clean sands and gravels to minimize siltation of the water. This sheeting will be removed near the end of construction. Design modifications to be made during subsequent phases of the project suggest that these requirements may be revised.

Groundwater

Slurry walls will be used on Connector CN-S at the Leverett Circle viaduct and from Causeway Street to Vent Building 5. Slurry walls will also be used northward to Station 164+90 at the mainline. Thus groundwater levels will be maintained at normal levels around the wood piles for the viaduct and the wood piles believed to be supporting buildings near Causeway Street. Groundwater pumping from soils will be minimized elsewhere because steel sheetpiling for excavation lateral support will typically be driven to glacial till.

Excavations for much of Connector CN-S will be extended relatively close to the Cambridge Argillite bedrock. As with the Central Area, pumping from the rock is anticipated to be required to limit hydrostatic uplift pressures below excavation subgrades. There are relatively few structures close to the alignment. Thus there is less potential in the North Area that pumping from the rock will result in adverse effects on building

foundations due to consolidation of clay strata. This is a matter requiring further evaluation in project design.

In order to provide for normal groundwater flow, slurry walls should be demolished to elevation 0 near the end of construction at intervals, as described previously for the Central Area. Permanent sheetpiling not forming seawalls should similarly be cutoff to elevation 0. Design modifications to be made during subsequent phases of the project suggest that these requirements may be revised.

Building Demolition

Anelex Building

The 13-story Anelex Building at 150 Causeway Street conflicts with the proposed location of the widened Central Artery southbound and associated new ramp structures. Other alignment constraints make it impossible to avoid this conflict. Therefore, it will be necessary to demolish the Anelex building.

Charles River Building

Connector CN-S will conflict with the west portion of the Charles River Building, at Beverly and Lovejoy Streets. The structure is actually four separate but adjoining buildings, built during the period 1901 to 1909. There are eight or nine stories, with a basement under some portions. Foundation conditions are unknown but presumed to be granite blocks supported on timber piles, based on soil conditions and the building age.

Underpinning this structure has been judged impractical, for the purposes of this study. Also, a space is needed for a ventilation building. Therefore, it will be necessary to demolish the western most two sections (about 100 ft.) of the Charles River Building to allow construction of Connector CN-S and to accommodate Vent Building 5. However, design modifications during Preliminary Design to minimize impacts to the Charles River's edge may also reduce impacts on this building.

5.5.5 South Boston (Station 37 to Station 92+30)

General

This section of the alignment involves the cut-and-cover construction of the portion of the Third Harbor Tunnel in South Boston beginning at the east side of the Fort Point Channel and continuing to the sunken tube at C Street. Included in this construction are the four ramps for the proposed Northern Avenue connector, and two ramps each to Summer Street and Northern Avenue near the Boston Marine Industrial Park. There will be three vent buildings: one near Northern Avenue; one between Viaduct Street and Summer Street; and one adjacent to A Street. As part of the project, the existing viaduct at Summer Street and a portion of the Viaduct Street viaduct will be reconstructed.

Foundations for Structures

Soil at the bearing level of the tunnel and most of the ramps generally consists of clay, glacio-fluvial and glacial till soils are considered suitable for support of the tunnel, ramp, and vent building structures; mat foundations are generally recommended.

Organic soil and fill will be encountered at subgrade level for segments of the ramps. Limited thicknesses of organic soil below subgrade level will be excavated and replaced with compacted granular fill, for support of ramp slabs and walls. In some areas of thick organic soils, pile foundations will be used to support ramps. Elevated ramps will also be supported on piles. Deep endbearing precast concrete or steel H-piles will probably be used.

When located over the new tunnel or ramps, piers and footings for reconstructed bridges for Summer

Street and Viaduct Street may be shallow supported on compacted granular fill. Tunnel structural design will consider these load concentrations. In areas not over the tunnel, it is recommended that end-bearing pile foundations be assumed for support of the reconstructed bridges in South Boston. Reconstructed bridges should be designed with simply supported spans as necessary to accommodate differential settlement arising from differing foundation types.

Lateral Support of Excavation

Station 37 to Station 86. Excavation depths will generally vary from 30 to 55 ft., with shallower excavations required for portions of the ramps. Interlocking steel sheetpiling will generally be suitable for lateral support of excavations. At A Street, where there are buildings immediately adjacent to the excavation, reinforced concrete slurry walls will be required to provide relatively rigid lateral support, to reduce movements of adjacent ground and buildings and to reduce ground vibrations that generally accompany installation of steel sheetpiling. Slurry walls will also be required to support the west side of the mainline tunnel excavation in the area of Turner and Carribou Fisheries to reduce movements of adjacent ground and buildings and to assist in providing access to these buildings during construction. Support for the sheetpiling and slurry walls may be either by tiebacks or internal bracing.

Proposed construction would remove the Summer Street embankment west to about the Right Way building, thus eliminating the need for lateral support of the street embankment.

Stability of the excavation bottom is generally considered adequate throughout this section. In areas underlain by thick clay deposits, some additional toe penetration of the sheetpiling will be required to improve bottom stability. The shallower excavations for ramps that will be adjacent to the deeper mainline tunnel will also serve to improve bottom stability.

Station 86 to Station 92+30. Excavation depths planned for this portion of the South Boston tunnel range from 50 ft. at Station 86 to 80 ft. at the interface with the sunken tube, at C Street. Because reinforced concrete slurry walls are stiffer and can support larger loads than steel sheetpiling, they will be used for lateral support of the sides of this deep excavation. The toes of the slurry walls will be stiffened and extended some additional distance below the bottom of the excavation or into glacial till to provide base stability in the deep excavations. Support for concrete slurry walls may be either by tiebacks or internal bracing.

Dewatering

In some areas, pumped pressure relief systems will be required to prevent uplift of excavation bottoms due to unbalanced hydrostatic pressures in the previous strata beneath the clay stratum. Sheetpiling or concrete slurry walls can be advanced into the pervious strata to reduce groundwater flow.

Dewatering or depressurization of deep granular strata and/or bedrock may induce consolidation in the clay. Several inches of long-term settlement could result. Piezometric levels should be monitored during construction. Recharging and/or grouting may be required to reduce the influence zone of excavation pressure relief.

Effects on Adjacent Structures

Existing structures very close to the proposed tunnel and ramps include a five-story brick building at 289 A Street, the five-story Bloom, South and Gurney Inc. building, the one-story North Coast Sea-Foods building, the Summer Street and Viaduct Street viaducts, and the Turner and Caribou Fisheries

buildings. Potential for settlement of these closest structures is discussed below. Other existing structures further from the construction will be affected to a lesser extent. Foundation information on the Gillette Co. tanks (above ground) located directly south of the tunnel was not available at the time of the study.

Corners of 289 A Street and the Bloom, South and Gurney Building are within 10 ft. of a proposed ramp. Building foundation type is not known, however, they are presumed to be supported either on caissons or timber piles which derive support in the upper portion of the clay layer. Reinforced concrete slurry walls will be used for excavation lateral support adjacent to 289 A Street to reduce ground movements. Little excavation is required adjacent to the Bloom, South and Gurney Building because the ramp in that area will be near existing grade; associated ground movements should therefore be negligible.

The six-story brick building at 259 A Street is also presumed to be supported on caissons or timber piles. It is on the edge of the influence zone of significant movements from the excavation, and is expected to experience only minor settlement due to lateral support system movements. However, slurry walls will be used to support the adjacent excavation to reduce ground vibrations which would be caused by sheet pile installation.

The North Coast Sea-Foods building, near Sta. 60, is about 30 ft. away from the adjacent ramp, and is outside the influence zone of significant movements from the excavation. Steel sheetpiling will be used for excavation lateral support in this area.

The Turner and Caribou Fisheries buildings are within 15 ft. of a proposed ramp and the toll plaza area. These buildings are presumed to be founded on spread footings although building foundation type is not known. Reinforced concrete slurry walls will be used for excavation lateral support adjacent to these structures to reduce ground movements and allow continued use of the facilities during construction.

Other existing structures along the alignment, such as the Hub Folding Box Company building on Wormwood Street and the General Ship building on 8th Street are outside the influence zone of significant movements from the excavation.

Based on measurements of movements on other projects, it is estimated that proper construction procedures can limit settlement of these adjacent buildings to approximately one inch. Published criteria indicate that these settlements should not cause appreciable structure damage.

The Summer Street viaduct and part of the Viaduct Street viaduct are to be removed and rebuilt under this project. Several piers of that portion of the Viaduct Street viaduct which will be retained are near the proposed construction. The piers are supported on timber piles driven into the clay stratum. Lateral support of excavation sides in this area will be by steel sheetpiles. Some adjacent ground movement will occur. Settlement of adjacent piers is expected to be tolerable and should not cause damage to the bridge structure, however, some adjustments to the bearings may be required.

Depressurization and dewatering of granular layers beneath the clay stratum may cause long-term settlement of ground adjacent to the excavation due to consolidation of the clay. Special procedures such as grouting of pervious layers and recharging may be required to limit the magnitude of expected settlement. The effects of dewatering on piezometric levels and ground settlement should be monitored to determine if procedures to mitigate the effects are required. Because the settlement zone of influence would

extend a significant distance from the excavation, abrupt differential settlements would not be expected.

Reconstruction of Viaducts over Tunnel

New piers and columns for reconstructed viaducts at Summer Street and Viaduct Street which are located over the tunnel may be founded on spread footings on compacted granular fill. Such footings should not be closer than 10 ft. from the outside edge of tunnel or ramp structures to keep the footings' zone of significant pressure increase within the compacted granular fill over the completed structures.

5.5.6 Boston Harbor

General

The alignment under Boston Harbor extends over a length of 4,100 ft. In South Boston, the tunnel originates at C Street and passes below the ship berth between Pier No. 5 (the east side of Drydock No. 4) to the west and the landfill at the Boston Marine Industrial Park (formerly Piers No. 4, 3, 2 and 1) to the east. The tunnel crosses the harbor to Bird Island Flats at Logan Airport.

Subsurface conditions along the alignment in Boston Harbor are not known because the alignment differs significantly from previously studied alternatives and no test boring information in the vicinity of this alignment was available. In the ship berth between Pier 5 and the Boston Marine Industrial Park, subsurface information was available.

Tunneling Method

Evaluation was made of alternative methods for constructing the tunnel below Boston Harbor. These methods consist of the bored tunneling method and sunken tube construction, briefly described as follows:

Bored Tunnel. Also referred to as a mined or driven tunnel, as applicable to this project, this method consists of driving a circular-shaped tunnel underground. The tunnel is advanced by soil excavation followed immediately by erection of steel or concrete lining plates to support the soil and seal the tunnel against water inflow. A fabricated shield is advanced at the tunnel. The shield contains the excavating equipment and is designed to provide temporary ground support in advance of lining placement. Bored tunnel construction requires vertical shafts at the ends of the line for equipment, material and personnel access. A bored tunnel alternate for the project would consist of twin, 38-foot outside diameter tunnels.

Sunken Tube Tunnel. Refer to Chapter 9.0 "SUNKEN TUBE TUNNEL CONSTRUCTION". More commonly used in Europe to date, this method is one where the structure is precast in large units of concrete or steel and concrete. The units are then floated in place, positioned in the bed of the waterway, and joined together. The units are generally laid in a dredged trench, which is then backfilled.

The reinforced concrete-type sunken tube measures approximately 98 feet wide by 28 feet high in cross-section, and a typical unit is approximately 500 feet long. A minimum five foot thick stone cover is planned for protection over the tunnel, and a two foot thick sand bed is planned below the tunnel.

Considering the advantages and disadvantages of each method as outlined below, it is concluded that the tunnel should be constructed by the sunken tube method since the bored tunnel method appears not feasible for the project.

The sunken tube method can accommodate the shallowest profile grade necessary for a suitable vertical alignment at each end of the tunnel. Excavation of the clay and glacial till soils can be readily accomplished with conventional dredging equipment. Underwater slope

stability problems are not anticipated. For quantity estimates, it is recommended that 1:1 side slopes be assumed in the clay and glacial till for the trench excavation.

Both clay and glacial till soils are found along the proposed bottom grade for the sunken tube tunnel. The glacial till soils are compact, with high strength and low compressibility. Glacial till will provide a very stable support for the tunnel and contribute negligible settlement to the tunnel during or following construction.

The clay strata will also provide satisfactory support for the tunnel. However, the clay will swell in response to vertical stress release due to the excavation. Swelling during excavation will be followed by consolidation and settlement as load is replaced on the clay by the tunnel and backfill. Post-construction settlements of the sunken tube tunnel bearing on the clay are estimated to range up to approximately three inches in magnitude. Settlements of the sunken tube tunnel should be relatively uniform except at connections to a slurry wall-supported ventilation building in South Boston. Information provided by Christiani and Nielsen regarding the concrete sunken tube sections indicates that joints can accommodate up to six inches of differential settlement.

By comparison with bored tunnel methods for this project, the sunken tube method is significantly less complex and technically demanding, and involves less risk of delays due to construction problems. The sunken tube method, however, involves substantially greater dredging and material disposal requirements than the bored tunnel method.

The bored tunnel method is not considered feasible due to major problems associated with excavation, dewatering and stabilization, limited soil cover and alignment. Regarding subsurface conditions, the profile considered for the twin bored tunnels encounters mixed face soil conditions consisting of clay overlying dense silty sand with gravel, cobbles and boulders (glacial till). As indicated by boring SH 202, bedrock is above the bottom grade for a bored tunnel on part of the airport alignment crossing. Mixed-face tunneling, especially in bedrock, presents major difficulties for excavation, dewatering, and stabilization in bored tunnels.

Based on previous experience, the glacial till contains numerous boulders exceeding 18 inches in size, the largest size which can be practically accommodated by modern tunneling machines. Reports of construction of the MBTA Blue Line tunnel in the harbor refer to boulders up to five feet in size. It is anticipated that the strength of the boulder rock precludes effective use of a crusher on the shield cutter head.

Therefore, soil and rock conditions are not feasible for use of either earth pressure balance or slurry shield tunneling machines. In addition, the 38-foot diameter tunnel size is approximately five feet larger than the maximum diameter soft-ground tunneling machines used in the world to date.

The alternative to controlled-face type tunneling machines is shield tunneling with compressed air. Compressed air tunneling for projects of this nature is becoming obsolete due to high labor costs. Air pressures up to the practical maximum limit of about 40 pounds per square inch (psi) would be required to stabilize the tunnel at depths up to 120 feet below sea level. Productive time per man would be limited to about two hours per eight hour shift, with the remainder of time required for decompression. Air pressures could be lowered by utilizing a dewatering system to reduce hydrostatic pressures in the glacial till stratum.

The tunnel profile grade at the East Boston end of all harbor crossing

alignments allows on the order of five to ten feet of natural cover above the bored tunnel. This cover is not sufficient for tunneling, since an approximate 30 to 40 feet thickness is recommended to prevent blow-out and to provide a stable roof and face. Dumping 30 feet of underwater fill on the harbor bottom to provide the necessary cover is not practical.

If the profile were lowered in the central portion of the harbor crossing, compressed air tunneling may be considered technically feasible for these portions of the alternatives. Nevertheless, placement of 10 to 20 feet of underwater fill would probably still be required to control blowouts. Also, deep-water shaft construction would be required at the transition points between bored and cut-and-cover tunnel sections on both ends of the crossing. Expensive cofferdam construction would be required between the land and the tunnel shafts. Depressing the profile along a major portion of the tunnel alignment to accommodate bored tunnel construction is not acceptable from a highway design aspect.

The bored tunnel method is not well suited to the horizontal alignment. The preliminary layout was designed with five feet of separation between tunnels. This is not feasible for construction of twin 38-foot diameter bored tunnels. To limit tunnel lining stresses and deflections, the preferred horizontal separation between tunnels is approximately one tunnel diameter.

Tunnel Type, Support and Excavation

Sunken tube construction is recommended as the most economical and technically feasible method for the tunnel, given the limited available data on subsurface conditions along the alignment.

Sunken tube construction will continue through the berth between Pier 5 and the Boston Marine Industrial Park and a short distance into Bird Island Flats so that the connection with the cast-in-place tunnel can be accomplished "on land".

Underwater excavation for the sunken tube will require some bedrock removal in the ship berth and possibly in Boston Harbor. Available test boring data in the ship berth indicates that bedrock is up to 35 ft. above the proposed tunnel subgrade. To remove the rock, blasting underwater would be required. A special drilling barge would be used to drill holes and set explosive charges. Controlled blasting techniques will be required to reduce ground vibrations to acceptable levels at the adjacent Drydock No. 4.

In Boston Harbor, bedrock may not be present at proposed tunnel subgrade. Glacial till and marine clay were encountered at subgrade along other previously studied alignments. At tunnel foundation level, the clay and glacial till soils and bedrock are satisfactory for support of the tunnel.

If bedrock were found to be at shallow depths all along the alignment in Boston Harbor, it may be more economical and technically feasible to drive deep tunnels through bedrock, instead of using sunken tubes. For preliminary planning, the requirements for such deep tunnels driven through bedrock include: 38 ft. diameter tunnel for each two lanes of traffic, minimum 20 ft. of rock cover over tunnel crown, minimum 20 ft. spacing between adjacent tunnels, driven by either drill and blast techniques or tunnel boring machine. Deep rock tunneling would probably require some mixed-face tunneling at either end as the profile rises on land. This type of tunnel construction would only be feasible if the entire Harbor Tunnel could be constructed by rock tunnel methods.

Excavation Lateral Support

Special lateral support systems for the sides of the excavation for the sunken tube tunnel will not be required except at the ends where it

will connect with the cast-in-place portions of the tunnel. Concrete slurry wall cofferdams will be employed for earth support for these short "on-land" sections. Lateral pressures on the earth support systems required for the "on-land" portions of sunken tube construction will be less than for dewatered excavations for cast-in-place construction. Support for these slurry walls will be by tiebacks and possibly above-water internal bracing.

Through the berth between Pier 5 and the Boston Marine Industrial Park, the open-cut trench excavation through overburden and bedrock can safely be made. Excavation side slopes in bedrock can be assumed to be 1H:4V. A minimum ten ft. wide bench around the top of the trench excavation is recommended. No special lateral suppot will be required for the cellular cofferdam at Pier 5 because it will be about 100 ft. away and is founded on glacial till and bedrock. It will be necessary to temporarily cutback or steepen the slope of the filled area along the Boston Marine Industrial Park to provide the 10 ft. bench, however, the resulting slope will not require added lateral support.

Effects on Adjacent Structures

The location for the proposed tunnel in the berth between Pier 5 and the Boston Marine Industrial Park was selected to minimize the impact of proposed tunnel construction in the adjacent areas. A large portion of the trench excavation for the sunken tube tunnel in the berth will be into bedrock. Soil deformations due to the excavation should be negligible.

The cellular sheet pile cofferdam of Pier 5 and the concrete structure of Drydock No. 4 are founded primarily on glacial till and bedrock at levels up to 40 ft. above the bottom of the trench for the sunken tube. However, Pier 5 is about 100 ft. away from the proposed trench for the sunken tube tunnel and is outside the influence zone of significant movement from the excavation.

East of the sunken tube trench, the landfill for the proposed container transfer facility at the Boston Marine Industrial Park is being completed. Around the perimeter of the landfill is a rockfill dike. Along the trench for the sunken tube, the outer slope of the rockfill dike will be temporarily steepened from 1.5H:1V to about 1.4H:1V to provide a minimum 10 ft. wide bench around the trench. The stability of the rockfill dike slope will not be significantly affected.

Underwater blasting will be required in bedrock. With properly controlled blasting techniques, vibrations at Pier 5 and Drydock No. 4 should be within tolerable limits.

5.5.7 Logan Airport

General

The proposed alignment in East Boston is located primarily within Logan International Airport and the Bird Island Flats area. Beyond the area adjacent to the Eastern Airlines hangar, the tunnel divides into five ramps for connections to the airport access roads and Route 1A. Cut-and-cover construction is planned for the tunnel and ramps. The airport access roads will be depressed into underpasses in the area of the connector road in front of the American Airlines hangar. There will be a vent building in the Bird Island Flats area at the edge of Boston Harbor.

The elevation of the top of bedrock along the tunnel alignment is not known. Throughout much of this area the proposed tunnel subgrade is deeper than the available subsurface information. Bedrock may occur within the excavation.

Foundations for Structures

Soil at the bearing level of the mainline tunnel is generally of glacial till; bedrock may also occur.

The ramp structures will bear primarily on clay soils. The clay and glacial till soils are suitable for support of the tunnel structures and therefore mat foundations are generally recommended.

Along various shallow ramp segments organic soils will be encountered at subgrade level. Limited thicknesses of organic soil below subgrade level will be excavated and replaced with compacted granular fill for support of ramp slabs. Ramps which overly thick deposits of organic soils and bridges will be supported on pile foundations. For preliminary consideration, end-bearing precast concrete of steel H-piles should be assumed.

Lateral Support of Excavation

Excavation depths for the mainline tunnel generally vary between 60 and 80 feet; for the ramps they vary between 30 and 70 feet. For the deeper cuts (greater than 50 ft.) and those which extend into glacial till, concrete slurry walls will be used for lateral support of the sides of the excavation because they are stiffer, can support larger loads than steel sheetpiling, and are more readily installed through glacial till. To provide base stability in the deeper excavations in clay, toes of the slurry walls will be stiffened and extended some additional distance below the bottom of the excavation or into glacial till. Sheetpiling is generally suitable for the shallower ramp excavations.

If the top of bedrock occurs above the proposed bottom of excavation, a slightly wider excavation will be required to provide a bench at the bedrock surface. Lateral support of the sides of the excavation in bedrock will be provided by rock bolts. Controlled blasting techniques would also be required to maintain the integrity of the bedrock bench.

Support of the sheetpiling and slurry walls may be by either tiebacks or internal bracing. In relatively wide excavations, it may be impractical to use internal bracing; however, use of tiebacks may be precluded where thick deposits of organic soils exist.

Dewatering

In the area between approximately Sta. 205 and 210, pumped pressure relief systems will be required for several ramp excavations to prevent heave of excavation bottoms due to unbalanced hydrostatic pressures in the glacial till stratum beneath the clay stratum. Sheetpiling or concrete slurry walls can be advanced into the glacial till to cut off groundwater flow.

In other areas, pumped pressure relief systems may also be necessary to depressurize pervious zones of bedrock which may occur below less pervious glacial till.

Effects on Existing Structures

Excavation for tunnels and ramps, and sheet pile driving may cause ground movements and vibration of adjacent areas with potential impact on nearby existing structures. Existing structures very close to the excavations for the proposed tunnels and ramps include the General Aviation Administration building (to be demolished) and a 12-story structure at the Hilton Hotel. Proposed structures of the Bird Island Flats Development will also be within the influence zone of movements from the excavation.

Reinforced concrete slurry wall will be used for excavation lateral support adjacent to these structures to reduce excavation-related ground movements and reduce vibrations associated with installation of the lateral earth support system.

The American Airlines hangar is 40 to 60 ft. away from the proposed Access Road underpass and is outside the zone of significant movements from the excavation. Steel sheetpiling

will be used for excavation lateral support in this area.

5.5.8 Seismic Effects on Tunnels

The silt, clay, and glacial till deposits which underlie the proposed tunnels and ramps are not considered susceptible to liquefaction, seismically induced settlements or slope instability for the design earthquake conditions. Potential earthquake induced deflections of the sunken tube tunnel are well within tolerable limits. Structural design will include lateral earthquake forces on foundation walls, and superstructure. On the basis of available data, soil liquefaction is not considered a problem relative to pile foundation design for elevated structures in the North Area.

Table 2

Test Boring Reports
(18 sheets)

GUILD DRILLING CO., INC.

OO WATER STREET EAST PROVIDENCE, R. I.

SHEET 1 OF 4
DATE
HOLE NO SB-1
LINE & STA.
OFFSET
SURF ELEV. 11.5

TO Haley & Aldrich, Inc. ADDRESS Cambridge, Mass.
PROJECT NAME Third Harbor Tunnel LOCATION South Boston, Mass.
REPORT SENT TO above PROJ. NO.
SAMPLES SENT TO " OUR JOB NO. 83-307

GROUND WATER OBSERVATIONS	Rods-"NW"	CASING	SAMPLER	CORE BAR
At ____ after ____ Hours	Type	HW-NW	S/S	NXD3
	Size I.D.	4" 3"	1 3/8"	
At ____ after ____ Hours	Hammer Wt	300#	140#	BIT
	Hammer Fall	24"	30"	Dia.

	Date	Time
START	4/28/83	
COMPLETE	5/4/83	
TOTAL HRS.		
BORING FOREMAN	K. Allen	
INSPECTOR		
SOILS ENGR.		

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		0'-1'6"	D	7	8	5			Black Cinders & fine to coarse Sand - FILL	1	18"	8"
								4'				
		4'-5'6"	D	8	9	10			Gray Clay - FILL	2	18"	3"
		9'-10'6"	D	2	2	3				3	18"	6"
								14'				
		14'-15'6"	D	2	1	2			Gray clayey Organic Silt, trace of cinders - FILL	4	18"	11"
								19' ±				
		19'-20'6"	D	11	21	30			Gray fine to medium SAND, little silt	5	18"	12"
								24'				
		24'-25'6"	D	12	19	27			Brown fine SAND, little silt & medium sand	6	18"	14"
								29'				
		29'-30'6"	D	8	8	8			Brown silty fine SAND, little clay in 1" layers	7	18"	10"
								33'				
		34'-35'6"	D	4	3	4			Gray Silty CLAY, trace of fine sand lenses	8	18"	15"
		39'-40'6"	D	3	2	3				9	18"	18"

GROUND SURFACE TO 34' USED 4" CASING. THEN 3" to 144' then Cored

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used	
trace	0 to 10%
little	10 to 20%
some	20 to 35%
and	35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density		Cohesive Consistency			
0-10	Loose	0-4	Soft	30 +	Hard
10-30	Med. Dense	4-8	M/Stiff		
30-50	Dense	8-15	Stiff		
50 +	Very Dense	15-30	V-Stiff		

SUMMARY:
Earth Boring 144'
Rock Coring 10'
Samples 30

HOLE NO SB-1

TOWN PRESS - EAST PROV

GUILD DRILLING CO., INC.

30 WATER STREET EAST PROVIDENCE, R. I.

SHEET 2 OF [illegible]
DATE
HOLE NO SB-1
LINE & STA.
OFFSET
SURF. ELEV.

TO ADDRESS
PROJECT NAME LOCATION
REPORT SENT TO PROJ. NO.
SAMPLES SENT TO OUR JOB NO. 33-307

GROUND WATER OBSERVATIONS			CASING	SAMPLER	CORE BAR
At	after	hours	Type		
			Size I.D.		
At	after	hours	Hammer Wt.		BIT
			Hammer Fall		

	Date	Time
START		a.m. p.m.
COMPLETE		a.m. p.m.
TOTAL HRS.		

BORING FOREMAN
INSPECTOR
SOILS ENGR.

LOCATION OF BORING

Depth	Casing Blows per foot	Sample Depths From – To	Type of Sample	Blows per 6" on Sampler From To 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev.	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness. Drilling time, seams and etc.	SAMPLE No.	Pen	Rec
		44'-45'6"	D	3	2	4			Gray CLAY, trace of silty fine sand lenses	10	13"	13"
		49'-50'6"	D	2	2	4				11	13"	13"
		54'-55'6"	D	2	4	2		54'±	Gray CLAY	12	13"	13"
		59'-60'6"	D	3	1	4				13	13"	13"
		64'-65'6"	D	2	3	4				14	13"	16"
		69'-70'6"	D	2	3	4				15	13"	18"
		74'-75'6"	D	2	1	2				16	13"	13"
		79'-80'6"	D	2	2	3				17	13"	13"

GROUND SURFACE TO USED CASING. THEN

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used
trace 0 to 10%
little 10 to 20%
some 20 to 35%
and 35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density
0-10 Loose
10-30 Med. Dense
30-50 Dense
50 + Very Dense

Cohesive Consistency
0-4 Soft 30 + Hard
4-8 M/Stiff
8-15 Stiff
15-30 V-Stiff

SUMMARY:
Earth Boring
Rock Coring
Samples

HOLE NO SB-1

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE, R. I.

SHEET 3 OF
DATE
HOLE NO SB-1
LINE & STA.
OFFSET
SURF ELEV

TO | ADDRESS
PROJECT NAME | LOCATION
REPORT SENT TO | PROJ NO.
SAMPLES SENT TO | OUR JOB NO. 83-307

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR
At ____ after ____ hours	Type			
	Size I.D.			
At ____ after ____ hours	Hammer Wt			BIT
	Hammer Fall			

	Date	Time
START		a.m. p.m.
COMPLETE		a.m. p.m.
TOTAL HRS.		

BORING FOREMAN
INSPECTOR
SOILS ENGR.

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From - To 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		84'-85'6"	D	2	2	4			Gray CLAY	13	18"	15"
		89'-90'6"	D	2	4	4				19	18"	13"
		94'-95'6"	D	3	3	3				20	18"	16"
		99'-100'6"	D	3	2	3				21	18"	13"
		104'-105'6"	D	2	1	2				22	18"	13"
		109'-110'6"	D	2	2	2				23	18"	16"
		114'-115'6"	D	1	1	3				24	18"	13"
		119'-120'6"	D	2	2	4				25	18"	13"

GROUND SURFACE TO ____ USED ____" CASING. THEN ____

Sample Type	Proportions Used		140 lb Wt. x 30" fall on 2" O.D. Sampler				SUMMARY:
			Cohesionless Density		Cohesive Consistency		
D=Dry C=Cored W=Washed	trace	0 to 10%	0-10	Loose	0-4 Soft	30 + Hard	Earth Boring
UP=Undisturbed Piston	little	10 to 20%	10-30	Med. Dense	4-8 M/Stiff		Rock Coring
TP=Test Pit A=Auger V=Vane Test	some	20 to 35%	30-50	Dense	8-15 Stiff		Samples
UT=Undisturbed Thinwall	and	35 to 50%	50 +	Very Dense	15-30 V-Stiff		HOLE NO SB-1

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE, R.I.

SHEET ___ OF ___
DATE ___
HOLE NO. SB-1
LINE & STA. ___
OFFSET ___
SURF ELEV. ___

TO ___ ADDRESS ___
PROJECT NAME ___ LOCATION ___
REPORT SENT TO ___ PROJ. NO. ___
SAMPLES SENT TO ___ OUR JOB NO. 33-307

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR
At ___ after ___ hours	Type			
	Size I.D.			
At ___ after ___ hours	Hammer Wt.			BIT
	Hammer Fall			

	Date	Time
START		a.m. p.m.
COMPLETE		a.m. p.m.
TOTAL HRS.		

BORING FOREMAN ___
INSPECTOR ___
SOILS ENGR. ___

LOCATION OF BORING ___

DEPTH	Casing Blows per foot	Sample Depths From – To	Type of Sample	Blows per 6" on Sampler 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		124'-125'6"	D	2	3	4			Gray CLAY	26	13"	13"
		129'-130'6"	D	Wt.	of Rods &	Ham.				27	18"	18"
		134'-135'6"	D	2	2	9		135'4"	+	28	18"	18"
		139'-140'6"	D	18	27	22			Gray clayey coarse to fine SAND, little coarse to fine gravel	29	13"	3"
					(300# Wt.)							
		143'6"-144'	D	120/3"	80/3"			144'	-GLACIAL TILL-	30	6"	4"
		144'-149'	C				Average Drill Time 3		Gray weathered ARGILLITE, occasional clay seams	C1	60"	31"
		149'-152'3"	C				6			C2	39"	26"
		152'3"-154'	C				6	154'		C3	21"	25"
									Bottom of Boring 154'			
									Installed 1½" Obs. Well at 28'6" - Use 10' Slotted & 20' Solid plus 3" Guard Pipe			

GROUND SURFACE TO ___ USED ___ CASING. THEN ___

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used	
trace	0 to 10%
little	10 to 20%
some	20 to 35%
and	35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density		Cohesive Consistency			
0-10	Loose	0-4	Soft	30 +	Hard
10-30	Med. Dense	4-8	M/Stiff		
30-50	Dense	8-15	Stiff		
50 +	Very Dense	15-30	V-Stiff		

SUMMARY:
Earth Boring ___
Rock Coring ___
Samples ___

HOLE NO. SB-1

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE, R.I.

TO	Haley & Aldrich, Inc.	ADDRESS	Cambridge, Mass.
PROJECT NAME	Third Harbor Tunnel	LOCATION	South Boston, Mass.
REPORT SENT TO	above	PROJ NO	
SAMPLES SENT TO	"	OUR JOB NO.	83-307

SHEET 1 OF 3
DATE
HOLE NO. SB-2
LINE & STA.
OFFSET
SURF ELEV. 9.5

GROUND WATER OBSERVATIONS
At 11' after ____ Hours
At ____ after ____ Hours

Rods-"NW"	CASING	SAMPLER	CORE BAR
Type	HW-NW	S/S	NXD3
Size I.D.	4" 3"	1 3/8"	
Hammer Wt.	300#	140#	BIT
Hammer Fall	24"	30"	Dia.

	Date	Time
START	4/27/83	a.m. p.m.
COMPLETE	5/6/83	a.m. p.m.
TOTAL HRS.		

BORING FOREMAN K. Allen
INSPECTOR
SOILS ENGR.

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		0'-1'6"	D	8	24	12			Dark Brown fine to coarse Sand & Gravel, some cinders FILL	1	18"	11"
								4'				
		4'-5'6"	D	7	19	29			" trace of brick - FILL	2	18"	3"
								8'				
								9'	Wash looks like Org. Silt			
		9'-10'6"	D	3	2	5			Blue Clayey SILT (very slightly organic)	3	18"	14"
								14'	Gray Clayey SILT (organic odor)			
		14'-15'6"	D	3	3	4				4	18"	18"
								18' ±				
		19'-20'6"	D	12	21	27			Gray silty fine SAND, occasional silty clay layers	5	18"	12"
								24'				
		24'-25'6"	D	4	5	5			Gray Silty CLAY	6	18"	15"
		29'-30'6"	D	3	4	5				7	18"	14"
		34'-35'6"	D	4	4	6				8	18"	15"
		39'-40'6"	D	3	3	3				9	18"	15"

GROUND SURFACE TO 24' USED 4" CASING. THEN 3" to 104' then Cored

Sample Type	Proportions Used	140 lb Wt. x 30" fall on 2" O.D. Sampler — Cohesionless Density	Cohesive Consistency	SUMMARY
D=Dry C=Cored W=Washed	trace 0 to 10%	0-10 Loose	0-4 Soft 30 + Hard	Earth Boring 104'
UP=Undisturbed Piston	little 10 to 20%	10-30 Med. Dense	4-8 M/Stiff	Rock Coring 10'
TP=Test Pit A=Auger V=Vane Test	some 20 to 35%	30-50 Dense	8-15 Stiff	Samples 22
UT=Undisturbed Thinwall	and 35 to 50%	50 + Very Dense	15-30 V-Stiff	HOLE NO. SB-2

GUILD DRILLING CO., INC.

?CO WATER STREET EAST PROVIDENCE R.I.

SHEET 2 OF 2
DATE
HOLE NO. 33-2
LINE & STA.
OFFSET
SURF ELEV.

TO
PROJECT NAME
REPORT SENT TO
SAMPLES SENT TO
ADDRESS
LOCATION
PROJ. NO
OUR JOB NO 33-307

GROUND WATER OBSERVATIONS
At ____ after ____ hours
At ____ after ____ hours

	CASING	SAMPLER	CORE BAR
Type			
Size I.D.			
Hammer Wt			BIT
Hammer Fall			

	Date	Time
START		a.m. p.m.
COMPLETE		a.m. p.m.
TOTAL HRS.		

BORING FOREMAN
INSPECTOR
SOILS ENGR.

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		44'-45'6"	D	3	2	4			Gray Silty CLAY, trace of fine sand partings	10	18"	18"
		49'-50'6"	D	3	1	3				11	18"	18"
								54' ±				
		54'-55'6"	D	2	3	4			Gray CLAY	12	18"	18"
		59'-60'6"	D	2	2	3				13	18"	18"
		64'-65'6"	D	2	2	2				14	18"	18"
		69'-70'6"	D	2	2	3				15	18"	18"
		74'-75'6"	D	2	1	2				16	18"	18"
		79'-80'6"	D	2	2	2				17	18"	18"

GROUND SURFACE TO ____ USED ____" CASING. THEN ____

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used	
trace	0 to 10%
little	10 to 20%
some	20 to 35%
and	35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density		Cohesive Consistency	
0-10	Loose	0-4	Soft
10-30	Med Dense	4-8	M/Stiff
30-50	Dense	8-15	Stiff
50 +	Very Dense	15-30	V-Stiff
		30 +	Hard

SUMMARY
Earth Boring
Rock Coring
Samples

HOLE NO 33-2

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE, R.I.

SHEET [illegible] OF [illegible]
DATE
HOLE NO SB-2
LINE & STA
OFFSET
SURF ELEV

TO ________ ADDRESS ________
PROJECT NAME ________ LOCATION ________
REPORT SENT TO ________ PROJ. NO. ________
SAMPLES SENT TO ________ OUR JOB NO. 33-307

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR
At ____ after ____ Hours	Type			
At ____ after ____ Hours	Size I.D.			
	Hammer Wt			BIT
	Hammer Fall			

	Date	Time	
START			a.m. p.m.
COMPLETE			a.m. p.m.
TOTAL HRS.			

BORING FOREMAN
INSPECTOR
SOILS ENGR.

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From 0-6	6-12	12-18 To	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		84'-85'6"	D	2	2	3			Gray CLAY	18	18"	10"
		89'-90'6"	D	2	2	2				19	18"	16"
								92'6"				
		94'-95'6"	D	24	18	20			Gray badly weathered ARGILLITE	20	18"	9"
		99'-100'6"	D	21	21	45				21	18"	15"
								104'				
		104'-107'	C						Gray ARGILLITE Fractured & Seamy	C1	36"	17"
		107'-110'	C							C2	36"	33"
		110'-114'	C							C3	48"	-0"
								114'				
									Bottom of Boring 114' Installed Observation Well at 19'			

GROUND SURFACE TO ________ USED ________ CASING THEN ________

Sample Type	Proportions Used		140 lb Wt. x 30" fall on 2" O.D. Sampler			SUMMARY
D=Dry C=Cored W=Washed	trace	0 to 10%	Cohesionless Density	Cohesive Consistency		Earth Boring
UP=Undisturbed Piston	little	10 to 20%	0-10 Loose	0-4 Soft	30 + Hard	Rock Coring
TP=Test Pit A=Auger V=Vane Test	some	20 to 35%	10-30 Med. Dense	4-8 M/Stiff		Samples
UT=Undisturbed Thinwall	and	35 to 50%	30-50 Dense	8-15 Stiff		HOLE NO SB-2
			50 + Very Dense	15-30 V-Stiff		

GUILD DRILLING CO., INC.

00 WATER STREET EAST PROVIDENCE R

SHEET 1 OF 3	DATE
HOLE NO. SB-3	LINE & STA.
OFFSET	SURF ELEV. 11.2

TO Haley & Aldrich, Inc. ADDRESS Cambridge, Mass.
PROJECT NAME Third Harbor Tunnel LOCATION South Boston, Mass.
REPORT SENT TO above PROJ NO.
SAMPLES SENT TO " OUR JOB NO. 83-307

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR
At ____ after ____ hours	Rods-"NW" Type	HW-NW	S/S	NXD3
	Size I.D.	4" 3"	1 3/8"	
At ____ after ____ hours	Hammer Wt.	300#	140#	BIT
	Hammer Fall	24"	30"	Dia.

	Date	Time
START	4/25/83	a.m. p.m.
COMPLETE	4/27/83	a.m. p.m.
TOTAL HRS.		
BORING FOREMAN	K. Allen	
INSPECTOR		
SOILS ENGR.		

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev.	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
								0'	Asphalt			
		6"-2'	D	17	20	17			Brown fine to coarse Sand, some fine to medium gravel, little silt - FILL	1	18"	9"
								4'				
		4'-5'6"	D	12	13	5			Brown Gray fine to coarse Gravel - FILL	2	18"	3"
								6'6"				
		9'-10'6"	D	1	1	1			Gray Organic SILT, trace of wood and fine sand	3	18"	12"
								14'				
		14'-15'6"	D	2	2	4			Gray Clayey SILT	4	18"	7"
								19'				
		19'-20'6"	D	7	12	17			Olive Silty CLAY	5	18"	9"
		24'-25'6"	D	7	10	14				6	18"	14"
								29'				
		29'-30'6"	D	5	6	9			Gray Silty CLAY	7	18"	14"
		34'-35'6"	D	3	4	6				8	18"	18"
		39'-40'6"	D	3	3	4				9	18"	18"

GROUND SURFACE TO ____ USED ____ CASING THEN ____

Sample Type	Proportions Used	140lb Wt. x 30" fall on 2" O.D. Sampler: Cohesionless Density	Cohesive Consistency	SUMMARY
D=Dry C=Cored W=Washed	trace 0 to 10%	0-10 Loose	0-4 Soft 30 + Hard	Earth Boring 52'2"
UP=Undisturbed Piston	little 10 to 20%	10-30 Med. Dense	4-8 M/Stiff	Rock Coring 10'
TP=Test Pit A=Auger V=Vane Test	some 20 to 35%	30-50 Dense	8-15 Stiff	Samples 13
UT=Undisturbed Thinwall	and 35 to 50%	50 + Very Dense	15-30 V-Stiff	HOLE NO SB-3

TOWN PRESS — EAST PROV

GUILD DRILLING CO., INC.

CO WATER STREET EAST PROVIDENCE R

SHEET 2 OF 3
DATE
HOLE NO SB-3
LINE & STA.
OFFSET
SURF ELEV.

TO
PROJECT NAME
REPORT SENT TO
SAMPLES SENT TO
ADDRESS
LOCATION
PROJ. NO.
OUR JOB NO. 33-307

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR
At ____ after ____ Hours	Type			
	Size I D			
At ____ after ____ Hours	Hammer Wt			BIT
	Hammer Fall			

	Date	Time
START		a.m. p.m.
COMPLETE		a.m. p.m.
TOTAL HRS.		
BORING FOREMAN		
INSPECTOR		
SOILS ENGR.		

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		44'-45'6"	D	2	3	3			Gray Silty CLAY	10	18"	15"
		49'-50'6"	D	2	1	4				11	18"	15"
		54'-55'6"	D	3	4	5				12	18"	12"
		59'-60'6"	D	2	2	4				13	18"	15"
								61'6"				
		64'-65'6"	D	24	24	22			Gray silty fine SAND, little clay, little fine to medium gravel -GLACIAL TILL-	14	18"	10"
								68'				
		69'-70'6"	D	9	15	20			Gray Silty CLAY, trace of fine to medium sand WEATHERED ARGILLITE	15	18"	4"
								74'				
		74'-75'6"	D	12	15	28			Gray badly weathered ARGILLITE	16	18"	12"
		79'-80'	D	75	175					17	12"	10"

GROUND SURFACE TO ____ USED ____" CASING. THEN ____

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used	
trace	0 to 10%
little	10 to 20%
some	20 to 35%
and	35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density		Cohesive Consistency		
0-10	Loose	0-4	Soft	30 + Hard
10-30	Med. Dense	4-8	M/Stiff	
30-50	Dense	8-15	Stiff	
50 +	Very Dense	15-30	V-Stiff	

SUMMARY:
Earth Boring
Rock Coring
Samples

HOLE NO SB-3

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE, R

SHEET 3 OF 3
DATE
HOLE NO SB-3
LINE & STA.
OFFSET
SURF ELEV.

TO
PROJECT NAME
REPORT SENT TO
SAMPLES SENT TO
ADDRESS
LOCATION
PROJ. NO
OUR JOB NO. 33-307

GROUND WATER OBSERVATIONS
At ___ after ___ Hours
At ___ after ___ Hours

	CASING	SAMPLER	CORE BAR
Type			
Size I.D.			
Hammer Wt			BIT
Hammer Fall			

	Date	Time
START		a.m. p.m.
COMPLETE		a.m. p.m.
TOTAL HRS.		

BORING FOREMAN
INSPECTOR
SOILS ENGR.

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From To 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
									Gray badly weathered ARGILLITE			
		83'-84'	D	36	115			84'2"		18	12"	9"
		84'2"-89'2"	C				4 Min/Ft		Gray ARGILLITE	C1	60"	50"
		89'2"-94'2"	C				2½ Min/Ft			C2	60"	49"
								94'2"				
									Bottom of Boring 94'2"			
									Installed Piezometer @ 66'			
									Installed Observation Well @ 16'			

GROUND SURFACE TO ___ USED ___ CASING. THEN ___

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used
trace 0 to 10%
little 10 to 20%
some 20 to 35%
and 35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density		Cohesive Consistency	
0-10	Loose	0-4	Soft 30 + Hard
10-30	Med. Dense	4-8	M/Stiff
30-50	Dense	8-15	Stiff
50 +	Very Dense	15-30	V-Stiff

SUMMARY
Earth Boring
Rock Coring
Samples

HOLE NO SB-3

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE R I

TO	Haley & Aldrich, Inc.	ADDRESS	Cambridge, Mass.
PROJECT NAME	Third Harbor Tunnel	LOCATION	Boston, Mass.
REPORT SENT TO	above	PROJ NO	4946
SAMPLES SENT TO	"	OUR JOB NO	82-272

SHEET 1 OF 3
DATE
HOLE NO SH-201
LINE & STA
OFFSET
SURF ELEV. -33.3

GROUND WATER OBSERVATIONS

At ______ after ______ Hours

At ______ after ______ Hours

Rods-"NW"	CASING	SAMPLER	CORE BAR
Type	6" - HW	S/S	
Size I.D.	NW	1 3/8"	
Hammer Wt	300#	140#	BIT
Hammer Fall	24"	30"	

	Date	Time
START	4/20/82	a.m. p.m.
COMPLETE	4/21/82	a.m. p.m.
TOTAL HRS.		
BORING FOREMAN	C.L., A.W.	
~~INSPECTOR~~	& J.P.	
SOILS ENGR.		

LOCATION OF BORING COORD: 494,934 N 726,410 E

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From 0-6	To 6-12	12-18	Moisture Density or Consist.	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc.	SAMPLE No	Pen	Rec
		0'-5.67'		P R	E S S	E D	(3")	1.8	Black SILT	E1	68'	-
							Wet					
		10'-11.5'	D	3	4	4	medium		Gray CLAY	1	18"	18"
							stiff					
	20	12'-13.7'	UP	P R	E S S	E D	(3")			U1	20"	20"
	17						Wet					
	26	13.7'-15.2'	D	wor	2	2	soft			2	18"	18"
	9											
	21	15.5'-17'	V Test									
	24											
	14											
	11											
	18											
	28	21'-22.5'	D	wor	1	3	"			3	18"	18"
	29											
	29											
	29											
	29											
	38											
	36	27'-29'	UP	P R	E S S	E D	(3")			U2	24"	24"
	39						Wet/m					
	45	29'-30.5'	D	3	3	5	stiff			4	18"	18"
	32											
	41	30.5'-32'	V Test									
	55											
	38											
	42											
	39											
	40											
	43	36'6"-38'	D	2	2	4	"			5	18"	14"
	40											
	45											

GROUND SURFACE TO ______ USED ______ "CASING THEN ______

Sample Type	Proportions Used	140 lb Wt. x 30" fall on 2" O.D. Sampler		SUMMARY
D=Dry C=Cored W=Washed	trace 0 to 10%	Cohesionless Density	Cohesive Consistency	Earth Boring 83'6"
UP=Undisturbed Piston	little 10 to 20%	0-10 Loose	0-4 Soft 30 + Hard	Rock Coring
TP=Test Pit A=Auger V=Vane Test	some 20 to 35%	10-30 Med. Dense	4-8 M/Stiff	Samples 13
UT=Undisturbed Thinwall	and 35 to 50%	30-50 Dense	8-15 Stiff	HOLE NO SH-201
		50 + Very Dense	15-30 V-Stiff	

TOWN PRESS — EAST PROV.

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE R I

SHEET 2 OF 3
DATE
HOLE NO SH-201
LINE & STA
OFFSET
SURF. ELEV.

TO
PROJECT NAME
REPORT SENT TO
SAMPLES SENT TO
ADDRESS
LOCATION
PROJ NO
OUR JOB NO 82-272

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR
At after Hours	Type			
	Size I D			
At after Hours	Hammer Wt			BIT
	Hammer Fall			

	Date	Time
START		a.m. p.m.
COMPLETE		a.m. p.m.
TOTAL HRS.		
BORING FOREMAN		
INSPECTOR		
SOILS ENGR.		

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From 0-6	To 6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
	41											
	46											
	40	42.5'-44.5'	UP	P R	E S S	E D	(3")		Gray CLAY	U3	24"	24"
	42											
	45											
	39	44.5'-46'	D	4	6	6	Wet stiff			6	18"	18"
	43											
	32											
	31	47.5'-49'	V Test									
	19											
	24											
	24											
	25	51.5'-53'	D	wor	4	5	"			7	18"	18"
	23											
	24											
	21											
	21											
	50							57'6"				
		57.5'-59'	D	35	30	18	Wet dense		Gray fine SAND, little clay, little fine to coarse gravel (Till)	8	18"	12"
		62.5'-64'	D	26	21	27	"			9	18"	8"
		67.5'-68.2'	D	87	100/3"		Moist very dense		Gray TILL & Cobbles	10	8"	4"
								72.5'				
		72.5'-72.8'	D	150/4"					White weathered ARGILLITE	11	4"	4"
								76.5'				
		77.5'-78'	D	103/3"	41/3" (300#)		"		" color change to Gray - Green DIABASE	12	6"	-

GROUND SURFACE TO ______ USED ______ "CASING. THEN ______

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used
trace 0 to 10%
little 10 to 20%
some 20 to 35%
and 35 to 50%

140 lb Wt. x 30" fall on 2" O D. Sampler

Cohesionless Density		Cohesive Consistency	
0-10	Loose	0-4	Soft
10-30	Med. Dense	4-8	M/Stiff
30-50	Dense	8-15	Stiff
50 +	Very Dense	15-30	V-Stiff
		30 +	Hard

SUMMARY:
Earth Boring
Rock Coring
Samples

HOLE NO SH-201

TOWN PRESS — EAST PROV.

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE R I

SHEET 3 OF 3
DATE
HOLE NO SH-201
LINE & STA
OFFSET
SURF ELEV

TO
ADDRESS
PROJECT NAME
LOCATION
REPORT SENT TO
PROJ NO
SAMPLES SENT TO
OUR JOB NO 82-272

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR		Date	Time
At ___ after ___ Hours	Type				START		a.m. p.m.
	Size I.D.				COMPLETE		a.m. p.m.
At ___ after ___ Hours	Hammer Wt			BIT	TOTAL HRS.		
	Hammer Fall				BORING FOREMAN		
					INSPECTOR		
					SOILS ENGR.		

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From To 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		83'-83.5'	D	160			Moist/v dense	83.5'	Gray weathered ARGILLITE	13	6"	-
				300#	Wt.				Bottom of Boring 83.5'			
									Grouted upon completion			

GROUND SURFACE TO ___ USED ___ " CASING: THEN ___

Sample Type	Proportions Used		140 lb Wt. x 30" fall on 2" O.D. Sampler				SUMMARY
D=Dry C=Cored W=Washed	trace	0 to 10%	Cohesionless Density		Cohesive Consistency		Earth Boring
UP=Undisturbed Piston	little	10 to 20%	0-10	Loose	0-4 Soft	30 + Hard	Rock Coring
TP=Test Pit A=Auger V=Vane Test	some	20 to 35%	10-30	Med. Dense	4-8 M/Stiff		Samples
UT=Undisturbed Thinwall	and	35 to 50%	30-50	Dense	8-15 Stiff		HOLE NO SH-201
			50 +	Very Dense	15-30 V-Stiff		

TOWN PRESS - EAST PROV.

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE R I

TO	Haley & Aldrich, Inc.	ADDRESS	Cambridge, Mass.
PROJECT NAME	Third Harbor Tunnel	LOCATION	Boston, Mass.
REPORT SENT TO	above	PROJ NO	
SAMPLES SENT TO	"	OUR JOB NO	82-272

SHEET 1 OF 2
DATE
HOLE NO SH-202
LINE & STA
OFFSET
SURF. ELEV. - 43.9

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR
At ______ after ______ Hours	Rods-"NW" Type	SW-HW	S/S	
	Size I.D.	NW	1 3/8"	
At ______ after ______ Hours	Hammer Wt	300#	140#	BIT
	Hammer Fall	24"	30"	

	Date	Time
START	4/22/82	a.m. p.m.
COMPLETE	4/23/82	a.m. p.m.
TOTAL HRS.		
BORING FOREMAN	A.W. - C.L.	
~~INSPECTOR~~	& J.P.	
SOILS ENGR.		

LOCATION OF BORING COORD: 494,731 N 725,273 E

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	Sample No	Pen	Rec
		0'-6.9'		P R	E S S	E D	(3")		Black SILT	E1	83"	0"
				Env. Tube								
								3.3'	(Tube broke off in hole)			
								6.5'	Gray stiff CLAY			
		9.5'-10.5'	D	27	78		Wet Hard		Gray SILT, little fine sand, gravel & clay (Till)	1	12"	12"
		15'-16'	D	38	52		"			2	12"	12"
				300# Wt.								
		21'-22.5'	D	20	27	80	"			3	18"	17"
				300# Weight								
		25'-27'	D	70	38	37	"			4	24"	13"
				300# Wt.		65						
		31'-32.5'	D	19	36	32	"			5	18"	13"
				300# Weight								
								35'				
		36'-37.5'	D	39	33	84	"		Gray SILT, little clay, trace of fine sand (Till)	6	18"	14"
				300# Weight								

GROUND SURFACE TO ______ USED ______ "CASING: THEN ______

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used	
trace	0 to 10%
little	10 to 20%
some	20 to 35%
and	35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density		Cohesive Consistency	
0-10	Loose	0-4	Soft
10-30	Med. Dense	4-8	M/Stiff
30-50	Dense	8-15	Stiff
50 +	Very Dense	15-30	V-Stiff
		30 +	Hard

SUMMARY:
Earth Boring 63'6"
Rock Coring
Samples 12

HOLE NO SH-202

TOWN PRESS - EAST PROV.

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE R. I.

SHEET 2 OF 2	
DATE	
HOLE NO SH-202	
LINE & STA	
OFFSET	
SURF ELEV.	

TO ____ ADDRESS ____

PROJECT NAME ____ LOCATION ____

REPORT SENT TO ____ PROJ NO ____

SAMPLES SENT TO ____ OUR JOB NO 82-272

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR
At ____ after ____ Hours	Type			
	Size I D			
At ____ after ____ Hours	Hammer Wt			BIT
	Hammer Fall			

	Date	Time
START		a.m. p.m.
COMPLETE		a.m. p.m.
TOTAL HRS.		
BORING FOREMAN		
INSPECTOR		
SOILS ENGR.		

LOCATION OF BORING ____

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From To 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
								41'	——— ——— ———			
		41'-42.5'	D	15	38	38	Wet Hard			7	18"	12"
				300# Weight								
		46'-47.5'	D	15	15	15	"		Gray clayey SILT with little very fine sand layers	8	18"	13"
				300# Weight								
								51'				
		51'-52.5'	D	32	38	27	"			9	18"	11"
				300# Weight				53.5'	Glacial Till			
		56'-57.5'	D	6	23	64	Dry very dense		White decomposed ARGILLITE	10	18"	12"
		61'-62.5'	D	28	45	61	"			11	18"	12"
		65'-65.5'	D	65			"	65.5'		12	6"	6"
		@ 65.5'	D	100/0"					Refusal - Bottom of Boring 65.5'			
									Hole Grouted			

GROUND SURFACE TO ____ USED ____ " CASING: THEN ____

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used	
trace	0 to 10%
little	10 to 20%
some	20 to 35%
and	35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density		Cohesive Consistency		
0-10	Loose	0-4	Soft	30 + Hard
10-30	Med. Dense	4-8	M/Stiff	
30-50	Dense	8-15	Stiff	
50 +	Very Dense	15-30	V-Stiff	

SUMMARY:
Earth Boring ____
Rock Coring ____
Samples ____

HOLE NO SH-202

TOWN PRESS — EAST PROV.

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE R. I.

TO Haley & Aldrich, Inc. | ADDRESS Cambridge, Mass.
PROJECT NAME Third Harbor Tunnel | LOCATION Boston, Mass.
REPORT SENT TO above | PROJ NO. 4946
SAMPLES SENT TO " | OUR JOB NO 82-272

SHEET 1 OF 1
DATE
HOLE NO SH-205
LINE & STA
OFFSET
SURF ELEV. -23.3

GROUND WATER OBSERVATIONS	Rods-"NW"	CASING	SAMPLER	CORE BAR
At ______ after ______ Hours	Type		S/S	
	Size I D	6"-4"	1 3/8"	
At ______ after ______ Hours	Hammer Wt	300#	140#	BIT
	Hammer Fall	24"	30"	

	Date	Time
START	4/28/82	a.m. p.m.
COMPLETE	4/29/82	a.m. p.m.
TOTAL HRS		
BORING FOREMAN	C.L. - A.W.	
INSPECTOR	& J.P.	
SOILS ENGR.		

LOCATION OF BORING COORD: 496,517 N 727,436 E

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc Rock-color, type, condition, hardness, Drilling time, seams and etc	Sample No	Pen	Rec
		0'-6'		P R	E S S	E D			Moved & took Env. 3" Tube	E1	72'	24"
									Black SILT	E2	24'	24"
								4.1'				
6"									Gray-Green Silty CLAY			
	71											
13.5'	148							8'				
	155											
in		10'-12'	D	27	35	34	Wet		Gray coarse to fine SAND	1	24'	4"
						41	very		& fine Gravel, little silt			
Ground							dense					
		15'-17'	D	30	16	31	Wet		Gray clayey SILT & fine	2	24'	20"
						26	Hard		Sand with Gravel embedded			
									(Till)			
		20'-21.5'	D	12	15	27	"			3	18'	18"
		25'-26.5'	D	27	30	40	"			4	18'	18"
		30'-32'	D	27	55	53	"			5	24'	24"
						70		32'				
									Casing broke 32'0"			
									Lost 20' - 4"			
									Lost 5' - 6"			
									Borehole not Grouted			

GROUND SURFACE TO ______ USED ______ "CASING. THEN ______

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used	
trace	0 to 10%
little	10 to 20%
some	20 to 35%
and	35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density		Cohesive Consistency	
0-10	Loose	0-4	Soft
10-30	Med. Dense	4-8	M/Stiff
30-50	Dense	8-15	Stiff
50 +	Very Dense	15-30	V-Stiff
		30 +	Hard

SUMMARY:
Earth Boring 32'
Rock Coring
Samples 5

HOLE NO SH-205

TOWN PRESS — EAST PROV.

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE R I

SHEET 1 OF 2
DATE
HOLE NO SH-205 A
LINE & STA
OFFSET
SURF. ELEV. -23.6

TO Haley & Aldrich, Inc. ADDRESS Cambridge, Mass.
PROJECT NAME Third Harbor Tunnel LOCATION Boston, Mass.
REPORT SENT TO above PROJ NO 4946
SAMPLES SENT TO " OUR JOB NO 82-272

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR
At ___ after ___ Hours	Rods-"NW" Type		S/S	
	Size I D	6"-4"	1 3/8"	
At ___ after ___ Hours	Hammer Wt	300#	140#	BIT
	Hammer Fall	24"	30"	

	Date	Time
START	4/29/82	a.m. p.m.
COMPLETE	4/30/82	a.m. p.m.
TOTAL HRS.		

BORING FOREMAN C.L. - A.W.
~~INSPECTOR~~ & J.P.
SOILS ENGR.

LOCATION OF BORING COORD: 496,518 N 727,436 E

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From To 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
								32'	Bottom of previous hole that broke off			
		35'-37'	D	14	18	19	Wet		Gray clayey SILT & fine Sand with Gravel embedded, cobbles (Till)	1	24"	24"
				300# Wt.		17	Hard					

GROUND SURFACE TO ______ USED ______ "CASING: THEN

Sample Type	Proportions Used		140 lb Wt. x 30" fall on 2" O.D. Sampler Cohesionless Density		Cohesive Consistency			SUMMARY	
D=Dry C=Cored W=Washed	trace	0 to 10%	0-10	Loose	0-4	Soft	30 + Hard	Earth Boring	71.1'
UP=Undisturbed Piston	little	10 to 20%	10-30	Med. Dense	4-8	M/Stiff		Rock Coring	
TP=Test Pit A=Auger V=Vane Test	some	20 to 35%	30-50	Dense	8-15	Stiff		Samples	8
UT=Undisturbed Thinwall	and	35 to 50%	50 +	Very Dense	15-30	V-Stiff		HOLE NO	SH-205A

TOWN PRESS — EAST PROV.

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE R I

SHEET 2 OF 2
DATE
HOLE NO SH-205 A
LINE & STA
OFFSET
SURF. ELEV.

TO
ADDRESS
PROJECT NAME
LOCATION
REPORT SENT TO
PROJ NO
SAMPLES SENT TO
OUR JOB NO 82-272

GROUND WATER OBSERVATIONS		CASING	SAMPLER	CORE BAR
At ____ after ____ Hours	Type			
	Size I D			
At ____ after ____ Hours	Hammer Wt			BIT
	Hammer Fall			

	Date	Time
START		a.m. p.m.
COMPLETE		a.m. p.m.
TOTAL HRS.		
BORING FOREMAN		
INSPECTOR		
SOILS ENGR.		

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From To 0-6	6-12	12-18	Moisture Density or Consist	Strata Change Elev	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		40'-41.5'	D	25	14	17	Wet		Gray clayey SILT & fine	2	18"	12"
				300# Weight			Hard		Sand with Gravel embedded,			
									trace of cobbles (Till)			
		45'-46.5'	D	11	14	21	"			3	18"	-
				300# Weight								
		50'-51.5'	D	12	18	24	"			4	18"	11"
				300# Weight								
		55'-56.5'	D	21	43	54	"			5	18"	8"
				300# Weight								
		60'-61.5'	D	25	35	52	"			6	18"	10"
				300# Weight								
		65'-66'	D	25	59		"			7	12"	10"
				300# Wt.				68'				
							Moist/v					
		70'-71.1'	D	58	121	50/1"	dense	71.1'	Gray Purple ARGILLITE	8	13"	11"
				300# Weight					Bottom of Boring 71.1'			
									Hole Grouted			

GROUND SURFACE TO ______ USED ______ " CASING: THEN ______

Sample Type	Proportions Used	140 lb Wt. x 30" fall on 2" O.D. Sampler — Cohesionless Density	Cohesive Consistency	SUMMARY
D=Dry C=Cored W=Washed	trace 0 to 10%	0-10 Loose	0-4 Soft 30 + Hard	Earth Boring
UP=Undisturbed Piston	little 10 to 20%	10-30 Med. Dense	4-8 M/Stiff	Rock Coring
TP=Test Pit A=Auger V=Vane Test	some 20 to 35%	30-50 Dense	8-15 Stiff	Samples
UT=Undisturbed Thinwall	and 35 to 50%	50 + Very Dense	15-30 V-Stiff	HOLE NO SH-205A

TOWN PRESS — EAST PROV.

TABLE 3

SUMMARY OF GROUNDWATER OBSERVATION READINGS

DATE	OW1 (SB1)	OW2 (SB2)	OW3 (SB3)	PZ1 (SB3)
	GROUNDWATER ELEVATION, FEET, (N.G.V.D.)			
	GROUNDWATER OBSERVATION WELL / PIEZOMETER (BORING)			
2 MAY 1983	-	-	4.5	0.2
10 MAY 1983	1.8	5.9	5.2	0.1
12 MAY 1983	1.6	5.5	5.3	-1.1
16 MAY 1983	1.8	5.4	5.6	
19 MAY 1983	2.3	6.0	5.8	0.9

INSTALLATION DATA (1)	OW1		OW2		OW3		PZ1	
	ELEV.	MAT'L	ELEV.	MAT'L	ELEV.	MAT'L	ELEV.	MAT'L
GROUND SURFACE	10.2	Fill	10.4	Fill	10.5	Fill	10.5	Fill
SEALS	9.7	Bentonite	7.6	Bentonite	3.5, 10.5	Bentonite	-49.0	Bentonite
POINT	-18.3	Sand	10.4	Sand	-5.5	Clayey Silt	55.5	Glacial Till
GROUTED ZONE	-143.8 to -24.8		105.4 to 15.4		--		46.0 to -6.5	
BOTTOM OF HOLE	-143.8	Argillite	-105.4	Argillite	-83.6	Argillite	83.6	Argillite

(1) DETAILS PERTAINING TO THE INSTALLATION OF OBSERVATION WELLS AND PIEZOMETER WERE REPORTED BY C. L. GUILD DRILLING CO., INC., EAST PROVIDENCE, RHODE ISLAND.

H&A FORM 6 – FEB. 74

TABLE 4

SUMMARY OF LABORATORY SOIL TEST RESULTS

FILE NO
SHEET 1 OF 1

BORING & SAMPLE NUMBER	DESCRIPTION	DEPTH (FEET)	TEST NO.	NATURAL WATER CONTENT %	ATTERBERG LIMITS % LL	ATTERBERG LIMITS % PL	UNIT WEIGHT LB/CU. FT.	UNCONFINED TEST COMPRESSIVE STRENGTH PSF	UNCONFINED TEST STRAIN %	CONSOLIDATION MAX. PAST PRESSURE TON/SQ. FT.	CONSOLIDATION C_c	OTHER TESTS
SH201/U1	Gray silty CLAY	12.0-13.7										
		12.1		39.9								TV = 0.4 PP = 0.7 SV = 0.4 R = 0.1
		12.3	UC13	41.1			109.9	1730	15.2			
		12.7	UC14	41.1			112.8	1280	4.2			
SH201/U2	Gray Silty CLAY	27 - 29										
		27.3		44.7								TV = 0.48 PP = 1.1 SV = 0.50 R = 0.14
		27.5	L2,UC15	42.9	52.5	23.0	111.9	1920	7.2			
		27.9	UC16	37.8			114.5	1450	3.4			
SH201/U3	Gray silty CLAY	42.5-44.5										
		42.7		40.7								TV = 0.53 PP = 1.4 SV = 0.6 R = 0.15
		43.0	UC17	41.8			112.2	1980	4.0			
		43.0	UC18	45.9			107.4	2420	1.8			

NOTES:
TV = Shear strength in TSF as measured by Torvane manufactured by Soiltest.
SV = Shear strength in TSF as measured by Lab Vane manufactured by Geonor.
R = Remolded shear strength in TSF as measured by Lab Vane manufactured by Geonor.
PP = Compressive strength in TSF as measured by Pocket Penetrometer manufactured by Soiltest.

TABLE 5

FIELD VANE SHEAR TEST RESULTS

BORING	TEST ELEVATION	SHEAR STRENGTH (PSF)	
		UNDISTURBED	REMOLDED
SH201	-48.8	559	79
SH201	-65.8	1621	1120
SH201	-80.8	442	88

NOTES:

1. Equipment: Field Vane

Type: Acker

Size: Diameter = 2.0 inches
Length = 4.0 inches

Torque Head Assembly

Type: Acker

Force Gage Type: Dillon

2. Field Vane Tests performed by Guild Drilling Co., Inc., and monitored by Haley & Aldrich, Inc., during the period 14-21 April 1982.

BIBLIOGRAPHY

R.B. Peck, A.J. Hendron, Jr. and B. Mohraz, "State-of-the-Art of Soft-Ground Tunneling", Proceedings, North American Rapid Excavation and Tunneling Conference, Chicago, Illinois, Vol. 1, June 1972, pp. 219-286.

Conduits, Culverts and Pipes, Department of the Army, Office of the Chief of Engineers, Engineer Manual EM 1110-2-2902, March 1969.

6.0 VENTILATION

6.1 DESIGN CRITERIA

Ventilation ducts and equipment conform to the ASHRAE criteria for minimum requirements for fire safety, 100 cubic feet per minute per foot of tunnel per lane. Full transverse ventilation has been assumed in all tunnel areas except as noted below. Tunnel ducts are sized to accommodate required air flows at a maximum velocity of 25 meters per second (56 miles per hour).

Nine ventilation buildings are proposed for this alternative. Vent-0 is located in the vicinity of the Massachusetts Turnpike - Central Artery Interchange; Vent-1 is located at Northern Avenue. Servicing the Depressed Central Artery will be: Vent-3 at Broad Street, Vent-4 at North Street and Vent-5 near Causeway Street. In South Boston Vent-7 is located on the Slade Gorton property off A Street near Melcher, Vent-8 near the intersection of Summer at Congress, and Vent-9 near Harbor Street at Northern Avenue. In East Boston Vent-10 is located in the Bird Island Flats, Phase 3 Development Area. Figure 89 depicts the approximate ventilation building locations for this alternative. It must be noted that the final locations for the ventilation buildings, particularly those serving the depressed Central Artery, can not be determined until additional detailed air quality analyses are performed to allow the concentrations of the emissions from the ventilation system to conform to federal and state air quality standards and policies.

6.2 TUNNEL DUCT SIZING

The "Tunnel Segment" designations of Table 6 are as follows:

The letters "A", "B", "C", "D" etc. refer to the tunnel segments or areas between adjacent ventilation buildings as depicted on Figure 89.

The "0", "1", "2", "3", ---- refer to the vent-building servicing that tunnel segment. (See Fig. 89)

"NBD" and "SBD" refer to Northbound and Southbound tunnels. Ramps or service roads regardless of direction have no "NBD" or "SBD" designation.

The building core size for each ventilation building (Table 7) refers to the size of the air shaft required to accommodate the combined maximum flow of air when all the air fans are operating.

Ventilating air from the common shaft to which all fans are directly connected is divided to serve the ducts of the various tunnel segments as listed in Table 6.

Except for tunnels served by Vent Building-8 the transverse system of air ventilation is to be employed throughout. In such a system fresh air is supplied at roadway level, and vitiated air is exhausted through ports in or near the ceiling. Supply and exhaust ducts may be below the roadway, above the ceiling or on the sides. In limited headroom areas (i.e. State St.) the exhaust and/or supply ducts were eliminated for short distances of tunnel in order to provide for the necessary vertical clearance between the ground surface and the crossing facility. In cases where the exhaust ducts are eliminated, the ventilation system becomes "semi-transverse". It is generally assumed that the air supply portion of the ventilation system will employ individually ducted segments with sensor operated baffles at the fan room to insure even air pressure throughout the entire tunnel length. Tunnels served by Vent Building-8 will be vented by a longitudinal system with exhaust fans in a shaft or adit near the center of the tunnel. Exhaust vitiated air will be drawn through the respective tunnels from their portals.

Table 6

TUNNEL DUCT SIZING

PREFERRED ALTERNATIVE

Alt.	Tunnel Segment	Length	No. of Lanes	Supply Air (KCFM)**		Duct Size (sf) Supply		Exhaust	
PREFERRED ALTERNATIVE	A-0-NB	300	4	120		24		24	
	B-0	1000	2	(200)		(40)		(40)	
	A-0	600	1	(60)		(12)		(12)	
	B-0	400	2	(80)	340	(16)	68	(16)	68
	B-0-NB	900	3	270		54		54	
	A-0	400	1	40		8		8	
	A-0	200	1	20		4		4	
	A-0	200	1	20		4		4	
	B-0	1200	1	(120)	210	(24)	42	(24)	42
	B-0-NB	300	3	(90)		(18)		(18)	
	J-0-SB	1000	2	(200)		(40)		(40)	
	J-0	400	1	(40)		(8)		(8)	
	J-0	400	2	(80)	320	(16)	64	(16)	64
	A-0	400	1	40		8		8	
	A-0	400	1	40		8		8	
	J-0-SB	1575	4	(630)		(126)		(126)	
	J-0-SB	800	1	(80)	710	(16)	142	(16)	142
	B-7-NB	500	3	150		30		30	
	B-7-NB	500	2	100		20		20	
	B-7	500	1	(50)		(10)		(10)	
	B-7	900	1	(90)		(18)		(18)	
	B-7	600	1	(60)	200	(12)	40	(12)	
	B-7-SB	600	2	120		24		24	
	B-7-SB	500	3	150		30		30	
	B-8-NB	700	2	210		L O N G I T U D I N A L			
	B-8-NB	700	2	210					
	B-8	550	1	85					
	B-8-SB	800	2	240					
	B-8-SB	600	2	180		T U N N E L			
	B-8	800	1	120					
	B-8	1300	1	195		V E N T I L A T I O N			
	B-9-SB	750	2	150		30		30	
	B-9-NB	750	2	150		30		30	
	C-9-SB	1900	2	380		76		76	
	C-9-NB	1900	2	380		76		76	

( ) Denotes not connected directly to Vent Building, i.e. connection via ductwork of other ramps or mainline tunnel.

Table 6 (continued)

TUNNEL DUCT SIZING

PREFERRED ALTERNATIVE

C-10-NB	1900	2	380		76		76	
C-10-SB	1900	2	380		76		76	
D-10	1500	1	(150)		(30)		(30)	
D-10	500	1	(50)		(10)		(10)	
D-10	1000	1	(100)	(300)	(20)	(60)	(20)	(60)
D-10-NB	2400	2	(480)		(96)		(96)	
D-10-NB	3200	2	(640)	1420	(128)	284	(128)	248
D-10	1600	1	(160)		(32)		(32)	
D-10-SB	400	2	(80)		(16)		(16)	
D-10-SB	900	2	(180)	(420)	(36)	(84)	(36)	(84)
D-10	700	1	(70)		(14)		(14)	
D-10-SB	3200	2	(640)	1130	(128)	226	(128)	226
J-1-NB	1575	4	630		126		126	
J-1	1400	1	150		28		28	
J-1-SB	200	4	80		16		16	
E-1-SB	350	4	140		28		28	
E-1	200	1	20		4		4	
E-1-NB	400	4	160		32		32	
E-3-NB	400	4	160		32		32	
E-3	300	2	60		12		12	
E-3-SB	350	4	140		28		28	
E-3	900	1	90		18		18	
F-3-NB	900	6	540		108		108	
F-3-SB	900	5	450		90		90	
F-4-NB	300	6	(180)		(36)		(36)	
F-4-SB	900	5	(450)		(90)		(90)	
F-4-NB	1000	4	(400)	580	(80)	116	(80)	116
F-4	700	1	(70)		(14)		(14)	
F-4-SB	300	4	(120)	640	(24)	128	(24)	128
G-4-NB	900	5	450		90		90	
G-4	1200	1	(120)		(24)		(24)	
G-4-SB	900	4	(360)	480	(72)	96	(72)	96
G-5	800	1	(80)		(16)		(16)	
G-5-NB	1100	5	(550)	630	(110)	126	(110)	126
G-5-SB	1100	5	550		110		110	
H-5	1000	1	(100)		(20)		(20)	
H-5	1000	2	(200)		(40)		(40)	
H-5	1800	2	(360)	660	(72)	132	(72)	132

( ) Denotes not connected directly to Vent Building, i.e. connection via ductwork of other ramps or mainline tunnel.

6.3 VENTILATION BUILDING SIZING

The ventilation building dimensions shown on Table 7 are preliminary and are shown in order to provide approximate building perimeter limits and to establish a basis for construction costs. The buildings are assumed to be four stories tall, except as noted, each story being 25 feet in height.

Carbon monoxide levels and the presence of smoke will be monitored in the Administration and Control Building, with fans being actuated to service the various tunnel segments as required. In the final design stage, cost and the "state-of-the-art" should determine whether control operations are manual or automated with a manual over-ride.

Three-speed fans rated at 350,000 cubic feet per minute have been utilized in all vent buildings. Each fan and its appurtenant motors require approximately 1200 square feet of floor area.

Table 7

VENTILATION BUILDING SIZING

PREFERRED ALTERNATIVE

Ventilation Buildings	Vent-0	Vent-1	Vent-3	Vent-4	Vent-5	Vent 7	Vent 8	Vent 9	Vent 10
SUPPLY AIR FLOW K-CFM	2160	1044	2196	1520	1840	720	1240	1060	3310
TOTAL AIRFLOW SUP & EXH K-CFM	4320	2088	4392	3040	3680	1440	1240	2120	6620
LOUVER INTAKE FT2									
@5 MPH	4910	2374	4991	3455	4180	1636	-	2410	7520
@7.5 MPG	3272	1582	3330	2300	2790	1090	-	1610	5015
BLDG CORE FT2	1500	700	1500	1000	1200	600	330	700	1900
CORE SIZE	75x20	60x12	75x20	60x18	60x20	24x24	22x22	60x12	75x25
TOTAL FANS	13	6	13	9	11	5	4	6	18
FANS/FLOOR	5	3	5	3	4	3	4	3	6
MECH ROOM FT2	6000	3600	6000	3600	4800	3600	5100	3600	6000
MECH RM SIZE	75x80	60x60	75x80	60x60	60x80	60x60	-	60x60	75x80
BLDG SIZE FT2	75x100	60x72	75x100	60x80	60x100	60x70	70x80	60x72	75x105
TOTAL BLDG FLOOR AREA FT2	30,000	13,000	30,000	19,200	24,000	12,600	11,200	13,000	31,500
STACK AREA FT2	1080	530	1080	760	920	360	620	530	1655
HEIGHT-STORIES OF 25 FT.	4	4	4	4	4	3	2	3	4

7.1 LIGHTING

7.1.1 Design Criteria

Tunnel lighting design criteria are based upon the following references:

a. Illuminating Engineering Society Lighting Handbook.

b. Design Report #14 for Fort McHenry Tunnel, Baltimore, Maryland.

c. IES transaction by A. Ketvirtis, FIES.

d. North Area/Central Artery proposed lighting scheme for the Massachusetts Department of Public Works.

e. U.S. Department of Transportation Roadway Lighting Handbook, December 1979.

There are numerous differences between the recommendations and conclusions of the references, but they all agreed on lighting all of the roadways included in this report and the following summations:

a. The illumination level maintained in the tunnel interior shall be 10 foot-candles (F.C.).

b. During daylight hours, a high level of illumination is required in the threshold zone (near the tunnel entrance) to allow for eye adaptation to the tunnel lighting levels ("Blackhole Effect" is created as a motorist enters the tunnel after travelling in daylight illumination (thousands of foot-candles) into the tunnel illumination (10-15 foot-candles).) This threshold illumination level varies considerably depending upon which standard is selected. (See Table 8.)

c. This threshold lighting can be reduced based on a ratio to access zone illumination (approach area preceding the entrance into the tunnel portal). This ratio also varies considerably between standards (See Table 8.)

d. The tunnel lighting from the threshold to the interior shall be reduced gradually at a 10:1 (IES) or 15:1 (U.S. DOT) ratio per zone in two or more zones (with 50 F.C. minimum if only 2 zones). The total length of these zones is based on human eye adaptation time of 8 seconds multiplied by the vehicles speed expressed in feet per second.

e. Nighttime lighting shall be uniform the full length of the tunnel (10 F.C.). (This will be achieved by switching off the daytime lighting fixtures. At the threshold zones this is done easily with several lamps per fixture. Where there is but one row of fixtures at reduced level transition zones, fixtures or lamps can be switched off leaving only the lamps necessary to produce a uniform illumination.)

f. The light source shall be linear, such as fluorescent and/or low pressure sodium, rather than a point source such as high pressure sodium, multivapor or mercury lamps.

7.1.2 Design Data

The following describes the design data and assumptions used in developing tunnel lighting concepts for the Third Harbor Tunnel project.

a. Tunnel speed for this application is based on 50 MPH (73+ feet per second) or 586 feet for 8 seconds. Each zone shall be no less than 3 seconds at 73+ feet/second = 220 feet.

b. Interior Tunnel Lighting = 10 F.C.; Threshold Tunnel Lighting = 500 F.C. For threshold lighting, 500 F.C. is used per IES recommendations (in lieu of the possibility of taking actual luminance levels at each access zone) which is a compromise between the Boston Central Artery North Area

Table 8

RECOMMENDATIONS FOR VEHICULAR TUNNEL LIGHTING

Authority	Threshold Illum. (F.C.)	Access Zone Illum. (F.C.)	Illumination Ratio Access vs. Threshold
AASHTO (1976)	500	***	***
IES (1972)	500	6565	13
CIE (1973)	1167	11,670	10
Japanese (62 mph)	146	5840	40
(50 mph)	131	5840	45
(37 mph)	88	5840	66
A. Ketvirtis (FIES)	*	**	25
U.S. DOT	*	**	10 to 15

* Computed according to a ratio of maximum ambient brightness level (or foot-candles) measured at each site.

** F.C. level equal to noon readings, June 21.

*** Not stated.

TUNNEL LIGHTING EXAMPLES

Tunnel	Location	Year Built	Threshold Foot-Candles
La Fontaine	Montreal, Canada	1967	100
Tuscarora	Pennsylvania Turnpike	1968	100
Thorald	Ontario, Canada	1968	80
Wallace	Mobile, Alabama	1973	200
Hampton Roads	Norfolk, Virginia	1976	81
Society Hill	Philadelphia, PA	1978	800
Ft. McHenry	Baltimore, Maryland	1980	215
Central Artery (North Area)	Boston, Massachusetts	not constructed	856

project and the Fort McHenry and Mobile, Alabama projects. It is recommended that actual readings be taken for the maximum conditions for final design. (Table 9 presents average foot-candle levels for the Boston area.)

c. The tunnel fixtures proposed shall be primarily fluorescent, utilizing eight-foot long fixtures with one, two or three tubes operating at 430, 800 or 1500 milliamperes as indicated in Figure 117, with supplementary 180 watt low-pressure sodium fixtures at threshold areas with tandem mounted lamps, one or two rows per fixture. Fixtures shall be watertight and enclosed, designed for washdown operation and using high power factor low temperature starting ballasts. Design voltage shall be 277 volts. Housings shall be die cast aluminum with hinged gasketed lens door. Lens shall be 1/8 inch clear prismatic, acrylic with smooth outer face. Fixture length allowance shall be 8'-6". For the purpose of calculating foot-candle levels to arrive at fixture quantities, coefficients of utilization were used from Westinghouse Electric PTC Series data. Three lamp fluorescent fixtures will utilize a two lamp ballast with a two lamp ballast on every other unit to service a single lamp in each fixture.

d. The lighting distribution system shall be 277/480 volts, 3 phase, 4 wire and served from power supplies originating at local ventilation substations. Threshold and transition lighting fixtures in the daytime could be controlled by the control center computer by time of day and ambient lighting sensing to conserve energy and turn off portions or all of the daytime light fixtures on dark, cloudy days. Emergency lighting shall be accomplished by means of an automatic diesel engine-driven generator set with automatic start and transfer facilities at each vent fan station. Emergency power shall be provided for the row of nighttime lighting fixtures on the walkway side only for a reduced level of lighting to five foot-candles.

7.1.3 Technical Data

Table 10 presents technical lighting data developed for the Third Harbor Tunnel project.

7.2. POWER DISTRIBUTION

The existing tunnel systems at the Callahan and Sumner Tunnels are supplied by seven Boston Edison Co. 13,800 volt feeders, four on the East Boston side and three on the Boston side. The two systems are not connected but there are some common Boston Edison Co. feeder numbers. There are no standby feeders as the total load is evenly distributed on all seven feeders. There are medium voltage (13.8 kV and 2.3 kV) tie feeders which run through the tunnels. Some of the switchgear is owned by the Massachusetts Turnpike Authority and others are owned by the Boston Edison Co. The present system has some complex cross connections requiring careful operations with key interlocking safeguards which entail manual action by an MTA or Boston Edison Co. operator to switch circuits.

The systems proposed for the Third Harbor Tunnel/Depressed Central Artery project should be simpler and automatic with as much or even more redundancy than the present schemes. The foremost consideration for the distribution system is reliability. There are numerous reliable schemes that have been developed over the years. Due to the length of the tunnel and the fact that all switchgear and substation equipment is limited to installation at the vent fan structures or outer ends, many of the schemes such as network or banked secondary can be automatically ruled out, and still others ruled out because of poor reliability factors. Two primary schemes lend themselves to this project; the first of these is a dual primary loop system with feeders from both ends and utilizing two air or vacuum circuit breakers at each transformer to loop through the system and back to the source. Pilot wire

Table 9

BOSTON AREA AVERAGE FOOT-CANDLE LEVELS HORIZONTAL*

	Dec. 21	March or Sept. 21	June 21
9 A.M. & 4 P.M.	100	2000	4700
10 A.M. & 2 P.M.	1600	4800	6800
Noon	2700	5800	7900

Average annual sunshine hours: 2400 out of 8760 Hours

Average hours of sunshine: Winter = 4-5 hours daily

Average hours of sunshine: Summer = 9-10 hours daily

Clear Days = 100-140; Cloudy Days = 120-160

* Based on IES Data - (Lat. 42°N)

Table 10

TECHNICAL LIGHTING DATA

		Init. Lumens	Lamp Watts	Hrs. Life
Lamps	430 ma. 8' fluor. T12	6300	75	12,000
	800 ma. 8' fluor. T12	9200	85	12,000
	1500 ma. 8' fluor. T12	15,500	212	10,000
	L.P.S.	33,000	180*	18,000

*241 W. at 18,000 hrs.

		QUANT. LAMPS			
Ballast & Lamp Watts		1	2	3	4
	430 ma. 8' fluor.	98	180	270	360
	800 ma. 8' fluor.	140	255	325	510
	1500 ma. 8' fluor.	230	450	565	900
	180 watt L.P.S.	220	440	660	880 initial
		(280	560	840	1120 @ 18,000 Hrs.)

relaying would be used to automatically isolate a bad piece of cable with no loss of power. Bus and transformer differential relaying would provide substation protection. Double ended substations with a normally open low voltage tie breaker and an automatic transfer scheme will be required for each substation. Two substations will be required at each vent fan substation due to a 4000 ampere limitation on the secondary equipment and the load will exceed this value for a single substation.

Primary selector type distribution system. A dual feeder at each side of the harbor and a third dual feeder at the northern end of the Central Artery will be required from the Boston Edison Company. The feeders will be sized such that any two feeders are capable of carrying the entire load on the system; however, in normal operation, the load will be shared among all feeders. The feeders will be 13,800 volts, three phase. Further redundancy can be obtained by connecting the two utility company services at each of the three distribution centers to incoming line circuit breakers in a 15 kV switchgear lineup, with the 15 kV bus split and connected through a fully integrated tie breaker which will be operated normally open. The feeder breakers will then be divided on both sides of the bus, and for still greater reliability, subdivided with a feeder breaker for each of the unit substations. Each of the substations will have the capability of being served from either side of the harbor or all from one side of the harbor. At each substation, the primary selector switches will be set to select one side fed from the southern source and the other from the northern source. With loss of power, either completely or partially, or on loss of a transformer, the secondary system will operate the same as with the primary loop system with automatic transfer utilizing the low voltage tie breaker to restore 100% service in seconds. Then if desired (if both transformers on either side of the substation are good) the primary selector switch could be transferred manually to feed both transformers from the same source, which would permit the tie breaker on the secondary to be restored to a normally open position. This restoration should be done manually, since primary switch selection would be done manually. The only advantage of the primary loop systems over primary selective is that in case of loss of any section of primary cable, the automatic secondary transfer scheme does not have to come into play at all, and there are no disturbances to the system.

The incoming feeders to each vent station will terminate in two double-ended unit substations. One unit substation in each station will have a secondary bus voltage of 2300 volts and will serve the large horsepower (300 HP and above) high speed fan motors. The second unit substation in each station will have a bus voltage of 277/480 volts and will serve the intermediate and low speed fan motors, tunnel lighting, pumping and building services, etc. Power at 208 volts or 120 volts for outlets, etc., will be derived by local, dry type transformers from the 480 volt system. Across-the-line motor starters or power air circuit breakers with motor overload protection for the fans will be incorporated into the switchgear lineup. Electrical interlocking will be provided to prevent more than one motor driving a fan and timers will be provided to prevent simultaneous restarting of fans after a power outage. Each unit substation will consist of two transformers, either of which at its forced air cooled rating could carry the entire load. A fully rated tie breaker, normally open, will connect the two switchboard busses. The loads will be split and fed from both sides of the switchboard. Main circuit breakers will be provided in the transformer secondaries. For the primary selector scheme, the primary of each transformer will be supplied either through two key-interlocked air interrupter selector switches which will each be supplied from opposite

sides of the harbor, or through medium voltage breakers as discussed on the primary loop scheme.

Load Calculations. Calculations assume that all fans are of the same capacity, and that there are three speed steps: 1/3 speed, 2/3 speed and full speed using a single high speed or one, two-speed motor. For example: a 350 HP single speed motor, operated at 2300 volts, will drive the fan at full speed, and a 100 HP two-speed variable torque motor, operated at 460 volts, will drive the fan at its two lower speeds. Also, for calculation purposes, 1000 cfm was used as equivalent to one horsepower, and 1 horsepower is equivalent to 1 kVA. The loads have been distributed evenly between the three proposed vent stations; however, in the final analysis, the loading apportionment could vary.

Table 11 presents the approximate load breakdown in terms of connected kVA.

Table 12 and Table 13 present energy requirements and costs.

7.3 ELECTRICAL DEFINITIONS

The following defines the items included and the basis for the electrical costs for the Third Harbor Tunnel/Depressed Central Artery project.

1. Switchgear - includes the cost to furnish and install all switchgear including three 15 kV double-ended switchgear assemblies to receive incoming Boston Edison lines and distribute to the vent buildings, a double-ended transformer type 2300 volt unit substation/motor control center for high speed vent fan motors in each vent building, a double-ended transformer type 480/277 volt unity substation for medium and low speed vent fan motors, tunnel lighting, pumping and building services in each vent building and a 480 volt motor control center for each vent building.

2. Primary Cable, Duct and Grounding - includes the cost to furnish and install primary conduits including spares interconnecting the unit substations with the 15 kV switchgear, all primary cables, and the complete grounding for the electrical distribution system.

3. Tunnel Lighting - includes the cost to furnish and install all lighting fixtures, lamps, conduit, wire, and distribution equipment. The price is per linear foot of one-way tunnel.

4. Tunnel Threshold Lighting - includes the cost to furnish and install all lighting fixtures, lamps, conduit, wire, distribution equipment, and automatic level control systems for one-way tunnel threshold.

5. Administration Facility - includes the cost to furnish and install materials for a complete lighting system, power distribution system, fire alarm system, connections to mechanical equipment, intrusion alarm system for a complete building electrical system for a 15,000 square foot building.

6. Ventilation Building Lighting and Power - includes the cost to furnish and install materials for a complete lighting system and power distribution system except wiring for ventilation fan motors for each vent building.

7. Pumping for Portal and Tunnel Low Point Drainage - includes the cost to furnish and install materials for lighting, pump controls, electric service from the pumps to the nearest vent building for each pumping station. Pumps and motors are not included.

8. Tunnel Traffic Control - includes the cost to furnish and install materials for lane control lights, conduit, and wire back to the control point and control equipment per linear foot of two-way tunnel.

Table 11

APPROXIMATE LOAD BREAKDOWN (kVA)

VENT BUILDING

ITEM	V_0	V_1	V_3	V_4	V_5	V_7	V_8	V_9	V_{10}	Admin.
Vent Fans-480V*	1200	1100	800	1300	1100	800	400	600	1800	-
Vent Fans-2300V*	4200	3850	2800	4550	3850	2800	1400	2100	6300	-
Pumping	183	183	183	183	183	183	183	183	183	-
Tunnel Lighting	827	827	827	827	827	827	827	827	827	-
Building Services	165	165	115	206	173	115	77	89	216	195
Total 480V Load	2375	2275	1925	2516	2283	1925	1487	1699	3026	195
Total 2300V Load	4200	3850	2800	4550	3850	2800	1400	2100	6300	-
Total Load	5375	5025	3925	5766	5033	3925	2487	3199	7526	195
								TOTAL LOAD	42,456 kVA	

* Not coincidental

Table 12

ELECTRICAL ENERGY USE IN BTU/YEAR

	kW-h	BTUx10^6
Ventilation	37,629,675	128,468
Tunnel Lighting	41,872,800	142,954
Building Services	3,603,650	12,303
Pumping	965,790	3,297
TOTAL	84,071,915	287,022

Table 13

TOTAL ENERGY USE PER YEAR

Ventilation	
Energy (kW-h)	37,629,675
Demand (kW)*	25,056
Tunnel Lighting	
Energy (kW-h)	41,872,800
Demand (kW)	7,444
Building Services	
Energy (kW-h)	3,603,650
Demand (kW)	1,515
Pumping	
Energy (kW-h)	965,790
Demand (kW)	1,354
Totals	
Energy (kW-h)	84,071,915
Demand (kW)	35,369
Costs (May 1982 Rates, $)	
Ventilation	
Energy	$2,427,112
Demand	1,177,416
Tunnel Lighting	
Energy	2,700,796
Demand	347,486
Building Services	
Energy	232,435
Demand	70,721
Pumping	
Energy	62,293
Demand	63,197
Totals	
Energy	5,422,636
Demand	1,658,820
TOTAL COST	$7,081,456

* Demand is based on the peak load/month (15 minute duration)

9. Telephone System - includes the cost to furnish and install telephones conduit and wire for local communication in tunnels and central telephone control. Also included are telephones for the vent buildings and for the administration facility.

10. Surface Intersection Traffic Signals - includes the cost to furnish and install traffic signals, poles, bases, controller, conduit and wiring for a typical four-way intersection.

11. Toll Collection Booth - includes the cost to furnish and install lighting, power, and connections to both the revenue counting devices and the power distribution system.

12. Fire Communication Repeater - includes the cost for antenna cable for two one-way tunnels, central repeater equipment, power supply and connections to telephone lines.

13. Lighted Sign - includes the cost to furnish and install back lighted signs, with conduit, and wire to the nearest ventilation building.

14. Induction Loop - includes the cost to furnish and install the detector unit, lead in conduit and cable, loop wire with saw cutting and pavement filling. Conduit for both power and signal wire will run to the nearest ventilation building.

15. Street and Roadway Lighting - includes the cost to furnish and install light fixtures, lamps, poles, bases, handholes, conduit, wire, excavation, backfill, and provide power sources for both City of Boston and Massachusetts DPW standards.

16. Vent Fan Wiring - includes the cost to furnish and install conduit, wiring, connections, and remote pushbuttons for vent fans and connections back to the motor controllers in the same vent building. The motor controllers are covered under Item 1 - Switchgear.

17. Underpass Lighting - includes the cost to furnish and install light fixtures, lamps and conduit and wire back to the nearest ventilation building.

18. Smoke Detection System - includes the cost to furnish and install duct mounted smoke detectors, conduit, wire and the central control panel in each tunnel section.

19. Carbon Monoxide Monitoring System - includes the cost to furnish and install detectors, conduit, wire, cable, controllers and recorders, in each tunnel section.

20. Closed Circuit Television System - includes the cost to furnish and install cameras, housings, conduit, wire, cable and monitoring equipment, in each tunnel section.

8.0 PROJECT COSTS

The total project costs, based on July 1982 prices, include the cost of construction, right-of-way, railroad work, and miscellaneous costs such as engineering and demolition of structures within right-of-way. The construction costs were obtained by developing preliminary estimates of the quantities of the various major items to be incorporated into the work, applying unit costs to these quantities, and making allowances for the numerous incidental items that will be developed from detailed plans. The unit costs used to develop the construction costs reflect the nature of the work and its location in a heavily built-up urban area. The construction costs include all work which must be performed to provide a complete tunnel facility and include the cost of all ramps, frontage roads, local street improvements, and the relocation of all major utilities made necessary by the proposed action. The estimated costs do not include work to be performed by others, such as the traffic management lane for the Southeast Expressway, the South Station Transportation Center, the new Northern Avenue Bridge, the Dewey Square TSM project, the North Area Project, etc.

Right-of-way costs were developed by certified appraisers; they include, land and building acquisition costs, demolition of existing buildings and premium costs for previously-planned developments (such as Boston Edison and Rowes and Fosters Wharf). These costs are based on fair market values and include a 35 percent factor for contingencies.

The cost of engineering and contingencies was taken as 10 percent of the total cost of construction.

Construction costs (Tables 14) are presented for major geographical sections of the proposed action for the Preferred Alternative and are further subdivided into major categories as follows:

1) TUNNEL STRUCTURES: Drydock facilities, structural concrete, reinforcing steel, structural steel, piles, test piles, pile load test, waterproofing, setting tunnel elements, structural fill, tunnel pump stations, oil water separators, pavement, walkways, finishes (refractory tile, etc.), traffic control, lighting, special systems (communication, television, fire alarms, CO monitoring etc.), drainage, water line and smoke.

2) VENTILATION BUILDINGS AND EQUIPMENT: Piles, structural concrete, reinforcing steel, building elements, HVAC items, mechanical items, electrical items, and tunnel ventilation system.

3) TOLL FACILITIES AND EQUIPMENT: Islands, booths, canopy, utility tunnel, administration building, maintenance and employee facilities, site work, mechanical items and electrical items.

4) LATERAL SUPPORT AND UNDER-PINNING: Sheet piles, shoring, jacks, soldier beams and lagging, cofferdams, under-pinning and slurry walls.

5) EARTHWORK: Excavation, dewatering, ordinary borrow, gravel backfill within cofferdams, and crushed stone for working mat.

6) DREDGING: Barge mounted clamshell excavation into scows and dumped 20 miles at sea.

7) GRADE SEPARATED STRUCTURES: (Retaining Walls, Boat Sections, Viaducts): Structural concrete, reinforcing steel, piles, test piles and pile load test, lighting, drainage, structural fill,

Table 14

PREFERRED ALTERNATIVE

PRELIMINARY CONSTRUCTION COST SUMMARY (IN MILLIONS)

CONSTRUCTION ITEMS	NORTH AREA	CENTRAL AREA	SOUTH BAY AREA	SOUTH BOSTON	BOSTON HARBOR	EAST BOSTON	PROJECT TOTAL
TUNNEL STRUCTURES	43	204	153	128	155	96	779
VENTILATION BUILDING & EQUIPMENT	7	17	11	18	-	11	64
TOLL FACILITIES & EQUIPMENT	-	-	-	4	-	-	4
GRADE SEPARATED STRUCTURES	87	18	85	44	-	17	251
LATERAL SUPPORT & UNDERPINNING	36	271	126	38	-	87	558
EARTHWORK	13	89	50	52	39	37	280
DREDGING	-	-	-	-	6	-	6
ROADWAYS AT GRADE	2	7	6	3	-	5	23
UTILITY RELOCATIONS	4	40	28	6	-	8	87
MISCELLANEOUS	14	33	80	7	3	15	152
CONSTRUCTION TOTAL	206	679	539	300	204	276	2,204
ENGINEERING & CONTINGENCIES (10%)	21	68	54	30	20	27	220
RIGHT-OF-WAY	43	7	24	44	-	22	140
TOTAL PROJECT COST	270	754	617	374	224	325	2,564

railing, waterproofing, structural steel, pavement and curbing.

8) ROADWAYS AT GRADE: Pavement, subbase, curbing, sidewalks, guardrail, fencing, safety features, drainage systems, striping, signing, signalization, landscaping, lighting, fine grading and compacting subgrade, grading and finishing.

9) UTILITY RELOCATIONS: Sanitary sewers, storm drains, water lines, steam lines, gas lines, power lines, telephone lines, telegraph lines, alarm systems, and temporary utility bridges.

10) MISCELLANEOUS: Mobilization, site preparation, maintenance of traffic including: temporary bridges, detours, construction signing and police protection; site preparation; railroad relocations and pedestrian overpasses.

11) RIGHT-OF-WAY: Land acquisition, building acquisition and demolition of buildings.

The breakdown of costs by geographical location are based on the following limits:

<u>South Bay Area</u>: Frontage Road at Perkins Trucking Company along Fort Point Channel to Northern Avenue (north) and A Street (south).

<u>Central Area</u>: Dewey Square to Causeway Street.

<u>North Area</u>: Causeway Street to Leverett Circle (west) and Gilmore Bridge (north).

<u>South Boston Area</u>: A Street to Northern Avenue.

<u>Boston Harbor</u> (sunken tube): Northern Avenue to Bird Island Flats.

<u>East Boston Area</u>: Bird Island Flats to Route 1A (north) and Eastern Airlines Terminal (south).

9.0 SUNKEN TUBE TUNNEL CONSTRUCTION

9.1 CONCRETE SUNKEN TUBE

9.1.1 Introduction

This study forms part of the environmental impact study of the Third Harbor Tunnel under Boston Harbor and is limited to that part of the tunnel which could be constructed as a reinforced concrete sunken tube. A steel sunken tube has also been considered, as discussed in Section 9.2.

9.1.2 Basic Assumptions

The concrete sunken tube for this alignment would extend from Station 92 + 00 to Station 132 + 00 (Mainline tunnel).

The following basic specifications were used for the study:

Soil Conditions

The results of geotechnical borings are presented in the form of subsurface profiles along the alignment. The different soil layers assumed are shown on Figure 86. Refer to Chapter 5.0, Geology and Soils, for a more detailed description.

Alignment and Geometrics

The alignment of the tunnel shall be such that allowance is made for a 1200-foot wide by 45-foot deep shipping channel.

A minimum of 5 feet of cover material plus an overdredge allowance of 2 feet shall be provided in the shipping channel over the top of the tunnel as protection. Beyond the reaches of the shipping channel the tunnel shall have a minimum of 5 feet of cover.

The tunnel shall generally provide two 26-foot roadways, comprising two 12-foot lanes with 1-foot curb offsets on the inner side and on the outer side.

The vertical clearance for the roadways shall be in accordance with Section 11.5 of the Design Criteria chapter of this report.

For the roadway curbs see Typical Sections in Chapter 2.0 of this report.

Each roadway shall have a 2-foot and a 4-foot wide safety walk. The top of the safety walks shall be approximately 2 feet 6 inches above the roadway surface and be acceptable for a future patrol car with modification.

The vertical clearance over the safety walks shall be at least 6 feet 6 inches.

The length of the tunnel sections shall be limited to approximately 500 feet.

Allowable Stresses and Materials

The design of structural steel elements shall conform to the provisions of the AISC Standards. The basic steel shall be ASTM A36 and steel with higher strength shall be used where required.

In general all connections shall be welded. Welds shall be in accordance with AASHTO Standard Specifications for Welding of Structural Steel Highway Bridges.

The design of all structural concrete except the roadway slab shall be in accordance with the ACI Specifications. The roadway slab shall be designed in conformance with AASHTO. Design strength for structural concrete shall be minimum of $f_c = 4{,}000$ psi. All reinforcing steel, except for stirrup steel, shall have a yield strength of $f_y =$ 60,000 psi. Stirrup steel shall have a yield strength of $f_y = 40{,}000$ psi.

Loads and Unit Weights

The unit weights used in determining the dead load are found in Section 11.6.5 and Table 18 of the Design Criteria.

For all computations the weight of concrete and water shall be selected in a manner which shall result in the more conservative design. Stability calculations shall be based on the net volume of the concrete less all voids for conduits, pipes, niches, structural and reinforcing steel.

AASHTO HS20-44 shall be used for roadway slab design and highway surcharge.

Soil loads shall be computed in accordance with the referred unit weights. The overburden pressures shall be calculated based on the assumption that the soil profile extends to the elevation of the original ground surfaces.

Horizontal earth pressure shall be calculated using a coefficient of lateral earth pressure of 0.30 for active earth pressure and 0.45 for at-rest earth pressure. The values are subject to the design backfill material gradation and verification during the subsurface exploration program.

Pressures exerted by water on the structure shall be computed using mean water elevations for Boston Harbor. A flood condition shall be investigated with extreme high- and low-water conditions at elevation 10.3 and elevation -7.0 respectively. Mean tide level shall be considered at elevation 0.0. Elevations are referred to the National Geodetic Vertical Datum.

Design Loading Conditions

The following loading conditions shall be considered as the basis for design.

For Tunnel-in-Place Loadings (normal stress allowance):

1) Maximum backfill, active lateral earth pressure, surcharge, high water at elevation 6.0.

2) Minimum backfill, at rest lateral earth pressure, low water at elevation -6.0.

For Flood Stage Loadings (33% overstress allowance):

1) Maximum backfill, active lateral earth pressure, high flood water at elevation 10.3.

2) Minimum backfill, at rest lateral earth pressure and low flood water at elevation -7.0.

Buoyancy and Flotation

Freeboard shall be calculated with the high unit weight of concrete and the low unit weight for water.

The amount of sinking ballast shall be determined such that the safety factor against buoyancy is at least 1.05 immediately after the tunnel section has been lowered into the trench.

The completed trench tunnel shall have a safety factor against buoyancy of at least 1.10 without the weight of backfill and cover in place. The completed tunnel section includes roadway finishes, safety walks and passageways.

Ventilation

The sizes of the ventilation ducts have been calculated based on the ventilated tunnel lengths corresponding to the distance between the ventilation buildings, placed as shown on the drawings, and on a fully transversal system of ventilation.

More detailed analysis on ventilation requirements for this

project is contained in Chapter 6.0 of this report.

Corrosion Protection

A minimum of 2 1/2 inches of concrete cover shall be provided for protection of the steel reinforcement against atmospheric corrosion.

The possibility of electrolytic and stray current corrosion shall be investigated and, if required, provisions shall be made to monitor corrosion activity and protect against it, for instance, by providing extra thickness of steel shell and/or buried anodes with impressed current where the steel is not protected by concrete.

9.1.3 Description

Structures

The sunken tube would be a reinforced concrete structure with a rectangular cross-section, provided with a watertight membrane. The typical cross sections are shown on Figure 34. The membrane on the bottom and the sides is made of 1/4 inch thick steel sheets and on the top of a 4-ply bituminous membrane. The top membrane is protected by a reinforced concrete slab.

It will be noted that the proposed placing of the ventilation ducts along the outer sides of the cross-section permits doors to be arranged in the middle wall, connecting the two roadways. These doors will be situated at about 250-foot distances from each other, and will provide egress for roadway users and access for rescue crews during traffic emergencies.

Figure 34 also shows how lighting, alarm boxes and acoustic treatment can be arranged.

A drainage basin with pumps will be situated at the deepest point of the roadway. The basin will be installed in the space of the ballast concrete under the roadway and the pumps may be placed within the wall space.

As the unit weight of the sunken tube with backfill usually will be nearly equal to the unit weight of the soil it replaces, the present settlement characteristics of the soil will not be radically changed after the tunnel is placed. The borings indicate that small differential settlements of the soil can be expected, and these settlements will not be greater than those met at other tunnel sites. Consequently, no special precautions will be necessary for the tunnel structure besides providing for simple articulation and dilatation joints at suitable spacings to counteract the effects of differential settlements and of longitudinal movements due to variations of temperature.

Another matter is the differential settlement between the sunken tube and its adjoining structures, as these may have completely different settlement characteristics. The adjoining structures may be, for instance, heavy ventilation buildings either causing greater settlements, if supported directly on the soil, or no settlements at all if supported on piles. A differential settlement at the end joints of the sunken tube will not be acceptable in the longitudinal alignment of the road. In such case, an articulated, watertight joint as previously used in similar cases is proposed.

This alignment contains two separate roadways with two lanes in each half of the tube. The dredged trench is assumed with side slopes as shown on Figure 34.

Method of Construction

The bottom of the trench will be approximately 20 feet wider than the width of the tunnel element and at a level approximately 2 to 3 feet below its underside. The sideslopes are an average of 1:1 in the clay and

2:1 in the organic soils (see Figure 91). The trench could be dredged by either a bucket dredger or a cuttersuction dredger with the capacity of working to a depth of 100 feet. (The bucket (or clam shell) dredger has been assumed for reasons discussed previously in this report.) The dredged material will be discharged into barges and transported to the disposal site.

Silt materials deposited in the trench after the dredging operation will need to be removed immediately before the start of the sinking operation.

The tunnel elements would be fabricated in a drydock. The only commercial drydock within the Boston area, large and deep enough for building the elements is located in South Boston. This dock, however, can hold only two elements at one time which is insufficient for a rational work schedule.

Attention was therefore directed towards finding an area where a suitable drydock could be established which would accommodate all of the elements simultaneously.

Several alternative dry dock fabrication sites were investigated, including sites in Connecticut and Massachusetts.

Each dry dock could be divided into two separate compartments in order that one can be flooded before fabrication is completed in the second, thereby allowing an earlier start to the placing of tunnel elements.

Figure 92 indicates a possible proposal for the drydock.

Most elements have a simple geometry and are made with plane and parallel bottom and topsides. The fact that some of the elements include vertical or horizontal curves is of little significance since the cost and time of construction will be identical to the straight sections.

When all elements are fabricated and provided with temporary bulkheads, rubber gaskets, access shafts, and ballast the dry dock would be flooded and a channel dredged into the dock.

Each element would then be floated by deballasting, and prepared for towing and sinking operations. Preparations will include trim ballasting, placing of towage, anchorage and suspension gear, access and alignment towers, sinking rig etc. All preparations will take place within the drydock or at a temporary jetty specially constructed for the purpose.

The elements would be designed to be self-buoyant with a small amount of freeboard. Towing the elements from the dry dock to the tunnel site requires some ballast to counteract any unbalance in their floating state, and partly to reduce their freeboard to four inches when fully fitted out for sinking. This creates a maximum draft of about 28 feet for the tube and a minimum water depth of about 30 feet for towing. This depth will allow one to two feet of clearance under the "keel". The existing minimum depth of 35 feet for the Boston main shipping channel has sufficient depth for safe transport of the elements to the tunnel site.

Tugs suitable for towing the elements are readily available in Boston Harbor.

Some restrictions must be imposed on shipping in the channel during the transport and sinking of those elements which are situated under the channel. Experience gained from actual construction of other sunken tube tunnels indicates restrictions to shipping of a few hours at a time.

An element ready for sinking is shown on Figure 93.

Prior to an element being brought to the tunnel site, temporary foundation blocks would be placed in

the trench and anchor points, buoys, etc., alongside the trench. The anchor points will require that a width of approximately 500 feet on either side of the tunnel centerline is made available for temporary use by the contractor.

When the element is moored and positioned, the amount of ballast water will be increased until the element's buoyancy is eliminated and a predetermined low weight, suitable for lowering it onto the supports has been reached.

By pulling the element towards the previous element, the softer part of the rubber gasket is compressed between the two end frames, thereby sealing the narrow space between the bulkheads of the two elements. By pumping the water from inside this space the outside hydrostatic pressure will further squeeze the rubber gasket and ensure watertightness. Water ballast would be added until the element obtains a safety factor against uplift of 1.05. The sinking equipment would then be dismantled and removed.

Continuation of the described procedure will leave a narrow gap between the last element and the end of the cut-and-cover tunnel. This gap will be closed by prefabricated forms placed under water by divers and sealed with rubber gaskets in order that the space inside the forms can be dewatered.

The joints would then be completed in reinforced concrete from inside the tunnel.

Sand would be placed under the element by special equipment either through outside pipes from a rig, running on the roof of the element or through pipes cast into the tunnel bottom. When the whole of the sand foundation is placed, the weight of the elements would be transferred from the temporary supports onto the permanent foundation. Temporary bulkheads can then be dismantled and the permanent ballast of concrete added, thereby increasing the factor of safety against uplift. The joints between elements would be provided with additional watertight seals from the inside of the tunnel.

After the elements are settled onto their final foundation, the trench would be backfilled for its entire length and a five foot layer of stones would be placed on top.

It is assumed that the land based portion of the tunnel will be constructed within two parallel walls of steel sheet piles, at least the width of an element plus 2 x 10 feet apart and braced against each other with steel lattice structures. The structures are designed to allow openings for sinking rigs and access towers and are positioned at least 3 feet above the high water surface level.

The bracing will not allow room for a standard dredger within the sheetwalls, therefore, excavation of the trench in that section will be done by clamshell. Foundation blocks, anchor points, etc. will be placed by a floating crane from outside the sheetwall.

The tunnel element will be hauled into position within the confines of the sheetwalls and anchored to anchor points fixed to the walls. The element at this stage will have less than one foot of freeboard, which leaves about two feet under the bracing for bollards, hatches and other fittings. Sinking rigs and access towers are placed on the element by a floating crane from outside the sheetwalls and the sinking operation is performed in a similar manner as described for the other elements.

If sand is to be placed under the element by a rig running on the roof of the element, the rig for this sheetwall section will need to be modified in order to work under the bracing.

Backfill will be placed by clamshell and the earth retaining structure will be dismantled and removed as backfilling progresses.

Construction Program

After mobilization, construction will start on the dry dock for tunnel elements and on the in-situ structure to which the first element is proposed to be joined.

As soon as the drydock is completed, construction of the tunnel elements will start. Most of the elements will be under construction simultaneously, but in various stages of completion.

During construction of the elements, the tunnel trench will be dredged except for a short section at both ends immediately in front of the cut-and-cover tunnels. When all of the elements are completed the dock will be flooded and an access channel dredged into the dock. Simultaneously, the dredged trench will be extended to the cut-and-cover tunnel, where the first element is to be joined.

The elements will then be prepared, placed, founded, and backfilled one by one.

Construction of the remaining cut-and-cover tunnel is scheduled such that it will be ready in time for joining with the last element.

A reasonable time for construction of the work included in the study will be 42 months. A preliminary construction program is shown on Figure 94.

9.1.4 Additional Data

Construction at the selected location appears to be favorable for the construction of a sunken tube.

The reinforced concrete tunnel tube as described above is similar to that of other sunken tube tunnels already in use elsewhere.

The fabrication of the elements can be handled by any well organized concrete contractor.

A separate contract may be required for the fitting out, erection of alignment towers, sinking, and founding of the elements, since the number of contractors in the United States capable of undertaking such work is more limited.

If required, a ventilation building could be placed on top of the sunken tube. In this case, the tube element would need to be designed with fittings to receive the ventilation building element with knockouts in the roof to accept future ventilation ducts, etc. The ventilation building would also need to be designed with sufficient, permanent buoyancy corresponding to permissible load on the tunnel tube.

Some aspects of the sunken tube tunnel construction, as described previously in this chapter, are, however, recommended for further study at a later design stage.

9.2 STEEL SUNKEN TUBE

9.2.1 Introduction

This study forms part of the environmental impact study of the Third Harbor Tunnel under the Boston Harbor and is limited to that part of the tunnel which could be constructed as a steel sunken tube. Section 9.1 discussed the concrete sunken tube option.

General

The cross section proposed is best described as a "cylindrical" section (see Figure 34) with a flat base slab and roof. Supply air will be conducted under the roadway slab and exhaust air through semicircular areas to the side of each roadway section.

The alternative to this "cylindrical" section would be a "binocular" section as used on the

Fort McHenry Tunnel in Maryland. This cross section would have a greater overall height as supply air ducts are located below the roadway and exhaust air ducts are located above the roadway.

The advantages of the cylindrical design are flatter profile grades, a savings in the dredging quantity, a probable savings in the steel shell weight, as well as a savings in the fabricated steel price. The disadvantages of the cylindrical section are the higher amounts of structural concrete required and the necessity of placing structural concrete in the fabricating facility to stiffen the tube section for towing. This could present a problem to some shipyards and would probably limit the number of fabricators bidding for the work.

The advantage of the binocular shape is its lower construction cost, lower amount of structural concrete, a minimum amount of forming, and a slightly more stable cross-section for handling. The "lay barge" required for this section would be less expensive, simply because the tube section is narrower. The disadvantage of the binocular section, however, is the overall height which results in steeper profile grades with greater speed reductions, and higher dredging and backfill quantities.

9.2.2 Preliminary Method and Construction Schedule

Major items concerning the steel sunken tube construction method are discussed below.

Dredging - dredging would be accomplished by a bucket-type dredge with disposal at sea. The cost estimate for steel tube construction has been based on ocean disposal of the dredged material.

Fabricate and Deliver Tubes - tube fabrication would be done (not necessarily in the Boston area) at a manufacturer's location (see 9.2.4), towed to the site, and outfitted at or near the jobsite (see 9.2.5).

Tow, Sink, Join - most of the tube sections would be placed on a screened gravel bed by a "lay barge", a catamaran-type vessel which supports the tube as it is lowered from two girders supported by the floats on either side. After placing the tubes, final ballast would be put on the tubes and backfilling can begin. Interior bulkheads and placing concrete for roadway slabs would follow in sequence.

9.2.3 Costs

The construction cost estimate (see Chapter 8.0) is for the limits as shown on the drawings. In general, the limits of the steel sunken tube portion of the structure are from ventilation building to ventilation building (i.e., from Boston Marine Industrial Park to Bird Island Flats). The limits for this study are Station 92 + 00 to Station 132 + 00.

Support of excavation limits have been approximated as shown in Section 9.2.6 of this report. The detailed limits and configuration finally to be used are subject to the verification of existing utilities, existing building foundations, subsurface soil structure, and proximity to existing roadways or trackbeds.

9.2.4 Fabricating Sites

Because it is possible to fabricate steel sunken tube sections in shipyards and tow them (with six to nine feet of draft) long distances, the tube manufacturing site is rarely located near the job site, but simply depends upon where the manufacturer is located.

Several possible tube manufacturers and fabricating sites are listed below.

1. Wiley Manufacturing
   Port Deposit, Maryland

2. General Dynamics Corporation
   Quincy, Massachusetts

3. Bethlehem Steel
   Sparrows Point, Maryland

4. Newport News Shipbuilding
   Newport News, Virginia

5. Sun Shipbuilding & Drydock
   Chester, Pennsylvania

6. Chicago Bridge & Iron
   Pascagoula, Mississippi

9.2.5 Outfitting Sites

Tube outfitting (the placing of structural and ballast concrete to get the tube ready to be placed) will be done as close to shore as possible to permit economical access to the work for construction personnel and materials. The problem that arises is the tube, depending on its overall height, will draw in excess of 25 feet of water just prior to placing. This limits some of the available waterfront space. Another problem of outfitting is the number of tubes which must be outfitted at any one time. For the Third Harbor Tunnel project, there will probably be between five and seven tubes in various states of outfitting, requiring an appreciable amount of tie up/outfitting area. With these facts in mind, the outfitting of the tubes for the project might be done in two locations. One of these will be a first stage outfitting which will place as much structural concrete in the tube as possible. Sites for this outfitting might be Massachusetts Port Authority piers in East Boston, (rehabilitation is required and included in the cost of the job) or the Massport bulkhead in South Boston. The second stage outfitting will include placement of concrete in the tubes to the point where they would be ready for placing (full draft). Concrete for this stage would ideally be ballast concrete (no forms). This outfitting will probably be done from a floating facility located over the dredged tunnel alignment, which eliminates the need for locating and tying up deep draft waterfront facilities.

9.2.6 Limits of Steel Sunken Tube

The limits of the tube approach are dependent upon the depth of excavation required, proximity of existing foundations of piling, and the obstructions and geological material in the area.

If the profile depth is relatively shallow (30 feet to 50 feet), the excavation limits resulting from dredging would be narrower and could permit an open cut situation where support of excavation is minimal or not required. (For more detail of the geological characteristics of the area, see Chapter 5.0 of this report.)

9.3 OTHER ALTERNATIVES

Two alternatives for the construction of a cross harbor tunnel have been discussed in this document; 1) the concrete sunken tube and 2) the steel sunken tube. This study does not preclude the use of other alternatives such as the binocular shape sunken tube used for the Fort McHenry Tunnel in Baltimore, Maryland. Other alternatives should be studied during the preliminary design phase of this project.

10.1 SUMMARY

This energy study will analyze the daily energy consumption of Alternatives 1 (No-Build) and the Preferred Alternative using average daily traffic figures projected for the year 2010.

The consumption of energy was calculated using "Energy Requirements for Transportation systems," U.S. Department of Transportation, Federal Highway Administration, Office of Environmental Policy, June 1980. The guidelines suggest a two-part approach involving both direct and indirect energy use.

Direct energy use involves the direct consumption of fuel based on traffic-related and facility-related factors. Traffic-related factors include volume of traffic, speed, traffic flow characteristics, and composition of vehicle types. Facility-related factors include grade, curvature, pavement condition, and stops (signs and signals). Indirect energy related to materials and operations required for construction, including transport, and the energy consumed by operation and maintenance of the facility. Any build alternative requires a substantial one time energy expenditure related to the construction materials, operations, and equipment. It will also require maintenance after construction. Indirect energy also includes that energy used to manufacture vehicles and consumed as a result of wearout and maintenance related to their operation. A proportionate quantity of the total energy required to manufacture and maintain vehicles must therefore be assigned to the project.

Table 16 provides the results of the energy analysis in terms of equivalent annual energy consumption by each alternative for the year 2010. Construction energy and vehicle indirect energy values have been pro-rated according to estimated "useful lives", in order to provide meaningful comparisons.

10.2 CONCLUSION

Table 16 indicates that both alternatives are essentially equal with respect to energy consumption. The Preferred Alternative results in induced traffic being attracted to the regional highway network. On an annual basis, this alternative consumes about 77,700 more barrels of oil than the No-Build Alternative, even though the traffic flow will be improved relative to the No-Build Alternative. This is due to the additional traffic induced by the Project. Due to uncertainties about the data at the current state-of-the-art, a difference of this magnitude is not significant.

It is important to note that total vehicle-miles traveled over the entire project area increased slightly under the Preferred Alternative relative to Alternative 1.

10.3 ADDITIONAL ANALYSIS

10.3.1 Traffic

Table 15 summarizes traffic data contained in the FEIS/FEIR.

Table 15

YEAR 2010 ANNUAL VEHICLE-MILES AND VEHICLE-HOURS

Alt.	Vehicle-Miles	Vehicle-Hours
1	3,302,900,000	150,090,000
Preferred	3,320,600,000	140,926,000

The following percentages were used in the computation of Direct Energy Vehicle-related Indirect Energy.

a) Breakdown of Annual traffic* by vehicle types**:

Cars	85%
2 Axle, 6 Tire Trucks	10%
Tractor Semi-Trailer Trucks	5%

* Annual Veh-miles from CTPS Computer Programs (Year 2010).

** References: DEQE, Mobile Source Emission Inventory Data (2010).

b) Breakdown of Cars and Trucks by fuel type (Year 2010):

Cars, Gasoline	80%
Cars, Diesel	20%
2 Axle, 6 Tire Trucks, Gas	78%
2 Axle, 6 Tire Trucks, Diesel	22%
Tractor Semi-Trailer Trucks, Gas	
Tractor Semi-Trailer Trucks, Diesel	50%

Reference: DEQE

Data presented in Table 15 represents Traffic Zones 1 and 2 as described in Appendix 4 of the SDEIS/DEIR. These zones are bounded roughly by Newton to the west, Malden and Revere to the north and Milton to the south. Within this area, overall vehicle miles have decreased with Alternative 5A Modified for reasons discussed in section 10.2 above. Vehicle hours of travel have decreased and therefore indicate an overall travel time savings approximately 7.0 percent over the No-Build Alternative. A report entitled <u>Third Harbor Tunnel Study</u> prepared in part by Vollmer Associates, Inc. in January 1981 analyzed the diversion of travel movements to the third harbor crossing from selected links in the network. This data yields an approximate annual savings of 16 million vehicle miles in cross-harbor travel if the project is built. This translates to an energy savings which is in the order of 0.7 percent. From vehicle-mile andvehicle-hour savings, a congestion factor was developed and applied to the Direct energy and vehicle-related Indirect energy for Alternative 1. The data in Table 16 for Alternative 1 reflects this adjustment.

10.3.2 <u>Indirect Energy</u>

The calculations for power requirements for Alternative 1 and the Preferred Alternative summarized in Table 16 include Ventilation, Tunnel Lighting, Building Services and Pumping requirements.

Peripheral energy change recognizes energy resources that are not used in any manner by the system itself. Rather, it addresses the potential effect that a transportation system may have on energy use and availability in 50 percent of the area it services.

Peripheral energy consumption effects will be negligible, since the project:

1. is located in a heavily built-up area with very little area left for induced development;

2. will not increase traffic volume by significant levels;

3. will not pre-empt use of other transportation modes;

4. will not cause a significant shift in population density; and,

5. will have a minimum encroachment upon land and structures currently used, or likely to be used for other purposes.

In computing construction-related energy consumption for use and comparison in this analysis, only the major energy-consuming activities were included. Materials quantities were obtained from the construction cost quantity estimate. See Table 17.

Table 16

ANNUAL EQUIVALENT ENERGY CONSUMED (BTU's)

Direct Energy	No-Build Alternative	Preferred Alternative
Cars	9.118×10^{12}	8.907×10^{12}
2-Axle, 6-Tire Trucks	3.638×10^{12}	3.556×10^{12}
Tractor-Semitrailer Trucks	3.018×10^{12}	2.949×10^{12}
Total, Direct Energy	1.577×10^{13}	1.542×10^{13}
Indirect Energy		
Vehicle-related	1.632×10^{13}	1.640×10^{13}
Facility Construction	3.564×10^{10}	4.633×10^{11}
Facility Maintenance	1.01×10^{9}	9.517×10^{9}
Power	0	2.870×10^{11}
Total, Indirect Energy	1.636×10^{13}	1.716×10^{13}
Total Energy Expended Annually:	3.213×10^{13}	3.258×10^{13}
Total Energy in Terms of Equivalent Barrels of Crude Oil per day:	15,177 Bbl	15,390 Bbl
% Energy Consumption compared with Alternative 1 (No-Build/Redecking)		+1.40

Table 17

SUMMARY OF CONSTUCTION-RELATED ENERGY CONSUMPTION (BTU)

Summary Useful Lives	No-Build Alt.	Preferred Alternative
* 100 Yr.	0	7.509×10^{10}
** 50 Yr.	2.178×10^{10}	7.025×10^{10}
*** 25 Yr.	1.386×10^{10}	3.174×10^{11}
**** 10 Yr.	0	5.39×10^{8}
TOTAL	3.564×10^{10}	46.33×10^{10}

* Tunnels, Tunnel Excavation, Ventilation Buildings

** Boat Sections, Bridges

*** Bituminous Concrete Pavement, Aggregate Subbase, Drainage Structures, Signing, Guardrail, Earthwork, Railroad Relocations

**** Landscaping

11.0 DESIGN CRITERIA

11.1 DESIGN CONTROLS

1. The major controls governing vertical alignment are:

    a) The future Boston Harbor Main Navigation Channel and the clearance requirements established by the Army Corps of Engineers. (1200' wide - 45' deep).*

    b) The Charles River Basin and MDC Boat Locks where clearance requirements have been established by the U.S. Coast Guard. (Vertical clearance no less than 30' between mean water level and the bottom of structure at the main boat lock.)

2. M.B.T.A. Red Line Tunnels in Fort Point Channel. (Piles for the cut and cover tunnel to be driven no closer than 20'; Vertical clearance between tunnels to be no less than 2').

3. Sea walls in Fort Point Channel (construct cut and cover tunnel no closer than 15').

4. Minimum Portal Elevation: Elevation 14.0 based on maximum high water investigation (Elevation 10.3 plus freeboard).

5. Horizontal Control: Massachusetts Coordinate System.

6. Vertical Control: National Geodetic Vertical Datum of 1929 (U.S.G.S.), (Mass. DPW).

7. The M.B.T.A. Blue Line tunnel at State Street, where the depressed Central Artery must pass over the subway tunnel. (Vertical clearance between tunnels to be no less than 2').

8. The M.B.T.A. Orange Line tunnel at New Chardon Street, where depressed Central Artery slurry wall will be built adjacent to the M.B.T.A. tunnel between Stations 153+00 and 155+00.

* A minimum of 5 feet of cover material plus an overdredge allowance of 2 feet shall be provided in the shipping channel over the top of the tunnel as protection. Beyond the reaches of the shipping channel, the tunnel shall have a minimum of 5 feet of cover material.

11.2 ROADWAY DESIGN

1. Number of Lanes in Each Direction

Tunnel:	2 Travel Lanes*
Depressed Central Artery:	4 Travel Lanes plus Weaving Lanes
Connectors:	2 Travel Lanes
Ramps:	1 Travel Lane plus 1 Breakdown Lane

* Split Alignment: 3 to 5 Travel Lanes

2. Roadway Widths

Roadway Widths	Desirable Min.	Absolute Min.
Mainline Travel Lane	12'	12'
Mainline Right Shoulder	10'	1'
Mainline Left Shoulder	4'	1'
Mainline Median (does not include shoulders)	7'	2'
Ramp	22'	22'
Major and Local Street Lane	12'	10'
Parking Lane	10'	8'
Major Street Median	16'	12'

3. Tunnel Approach Roadways

2-12' Travel Lanes, each with an outside shoulder 2'.

4. Tunnel (Cut and Cover - Sunken Tube)

2-12' Travel Lanes, each with an outside shoulder of 1'.

5. Depressed Central Artery

4-12' Travel Lanes, 1 or 2-12' Weaving Lanes (where required) with 1' outside shoulders.

11.3 DESIGN SPEED (MPH)

Roadway	Desirable Min.	Absolute Min.
Main Roadway	70	50
Tunnel	70	50
Major Cross Street	35	25
Ramps	40	25
Local Streets	35	20
Tunnel Ramp	35	20
Depressed Central Artery	50	35

11.4 HORIZONTAL ALIGNMENT

1. Curvature - Compound Transitions Required

Design Speed (MPH)	Radii (Feet) Desirable Min.*	Absolute Min.**
20	360	120
25	550	200
30	800	275
35	1100	400
40	1400	525
50	2000	850
60	3000	1250
70	3200	1900

* Based on 4 percent cross slope
** Based on 6 percent cross slope

2. Curvature in Tunnels

Design Speed (MPH)	Min. radii in tunnels (feet) 2' safety walk	4' safety walk
20	250	200
30	600	500
40	1200	950
50	1900	1500

3. Narrowing of Roadways

A transition length of 1500 feet is desirable to effect a narrowing of one lane width at a design speed of 70 miles per hour and a transition length of 1200 feet is desirable for the narrowing of one lane width at a design speed of 50 miles per hour.

11.5 VERTICAL ALIGNMENT

1. Profile Grades

	Profile Grades (%) Min.	Desirable Maximum	Absolute Maximum
Mainline	0.5	3	6
Major Cross Street	0.5	4	5*
Ramp	0.5	5	6**
Other Roadways	0.5	5	6*
Railroad	-	1.5	3

* Or equal to existing grade

** Not attainable in the following locations:

Location	Gradient
South Bay Ramps (SS-T) (AT-SS) (C-T)	7%
Central Ramps (SA-CT) (ST-CN)	7%

2. Vertical Clearance

a.	Tunnels	16' - 6"*
b.	Open Roadways	14' - 6"
c.	Railroads	22' - 6'**
d.	Charles River Basin	30' - 0" ***

* 2' - 0" allowance over minimum clearance of 14' - 6" (existing vertical clearance for approach roadways) for signs, traffic signals, lighting and superelevation.

** 16' - 6" proposed to match existing vertical clearance. DPU approval will be required.

*** 30' clear at main lock.

3. Minimum Length of Vertical Curve

MPH V	Minimum Horizontal and Vertical Stopping Sight Distance		Length of Vertical Curve Values of "K" x A Sag Des.		Crest Des.	
	Min.	Des.	Min.	Des.	Min.	Des.
15	88		11A		11A	
20	125	150	19A	24A	15A	16A
25	162	175	27A		19A	
30	200	200	36A		28A	
35	240	250	46A		40A	
40	275	300	56A	60A	54A	65A
50	350	450	75A	100A	88A	145A
60	475	650	109A	155A	162A	300A
70	600	850	144A	215A	257A	515A

4. Reasonable Guide Values for Critical Length of Grade
(Based on 1973 AASHTO Speed Reduction of 15 MPH Desirable)*

Upgrade	3%	4%	5%	6%
Maximum Length	1600'	1100'	800'	650'

* During the EIS preparation guidelines for speed reduction were not attainable in the following locations; however, design modifications to be made during the Preliminary Design phase will improve overall speed reductions.

Location	Theoretical Speed Reduction
Central Artery N.B. 88+00 - 97+00	17
I-93 N.B. 163+00 - 178+00	37
Ramp 1A-1 218+00 - 233+00	24

11.6 STRUCTURAL DESIGN

1. Building Codes, Standards and Specifications

The design of the tunnel facility and adjacent ancillary structures shall be performed in accordance with the following current design codes, standards and specifications where applicable:

a) American Concrete Institute Building Code
b) AASHTO Standard Specifications for Highway Bridges
c) AISC Manual of Steel Construction
d) The BOCA Building Code
e) Commonwealth of Massachusetts State Building Code
f) AREA Manual for Railway Engineering, Chapter 8, Concrete Structures and Foundations
g) ASTM Standards
h) OSHA Standards
i) AASHTO Standard Specifications for Welding of Structural Steel Highway Bridges

The climate temperature variation used in the design shall be a low of -30°F and a high of 120°F for metal structures and a temperature rise of 35°F and a fall of 45°F for concrete structures.

2. Prefabricated Tunnel Sections

Criteria pertain to the design of the trench tunnel structure for all stages of construction (fabrication, launching, outfitting, placing) and in the final backfilled position. (Refer to Chapter 9.0 Sunken Tube Tunnel Construction of this report.)

3. Cut and Cover and Open Approach Structures

The project includes cut and cover and open approach construction along the respective alignments. The basic material for the structures shall be reinforced concrete.

4. Allowable Stresses and Materials

a) Structural Steel

Design stresses shall conform with provisions of AASHTO Standards.

b) Reinforced Concrete

Design stresses shall be in accordance with AASHTO Standards. The design shall be by the service load method. The concrete shall have a compressive strength (f'c) equal to 4,000 psi. All reinforcement steel shall have a yield strength (fy) of 60,000 psi, except that stirrups and ties have a yield strength of 40,000 psi.

c) Foundation Soil Stresses

Refer to Section 11.7 - Geotechnical Engineering, of this Chapter.

d) For temporary support of existing structures, an overstress of up to 1/3 will be allowed.

5. Loads and Unit Weights

a) The dead loads shall be determined using the following unit weight:

Material	Unit Weight (pcf) Max.	Min.
Structural Steel	490	-
Concrete, reinforced	150	145

For soils, refer to Table 18, Section 11.7 - Geotechnical Engineering, of this report.

b) Live Loads

AASHTO HS20-44 shall be used for roadway slab design and highway surcharge.

All railroad surcharge, where applicable, shall be Cooper E80.

c) Soil Loads

Refer to Table 19, Section 11.7 - Geotechnical Engineering of this report.

d) Design Surcharge

Surcharge shall be Highway Live Load, or Railroad Live Load where applicable.

6. Design Loading Conditions

a) Long Term Loading:

Refer to Geotechnical Engineering, Section 11.7 of this report.

b) During Construction Loading:

No groundwater.

c) Flood Stage Loading:

Flood water at El. 10.3, design surcharge: minimum buoyancy safety factor 1.05.

d) Uplift Stability Load:

Groundwater (See Section 11.7) neglect side friction: minimum buoyancy safety factor 1.15.

11.7 GEOTECHNICAL ENGINEERING

1. Load and Unit Weights

a) Dead Load.

The following unit weights shall be used in determining the dead load:

	Max.	Min.
SOIL	(See Table 19)	
Water (PCF)	64	62.5

For all analyses the unit weights for soil(s) and water shall be selected in a manner which shall result in the more conservative estimate.

b) Soil Load.

Soil Loads shall be computed in accordance with the unit weights given in Table 19.

In estimating lateral earth pressure, coefficients of lateral earth pressures given in Table 19 shall be used. Tables 21 and 22 present the allowable soil bearing pressures and the modulus of subgrade reactions for various soil types, respectively.

c) Design Groundwater Levels.

Design groundwater levels on land are assumed as follows:

AREA	DESIGN GROUNDWATER ELEVATION, FT., (NGVD)
South Bay	+ 6.0
Central	+ 6.0
North	
Central Artery, Sta. 164 to 170	+ 10.0
Central Artery, Sta. 170 to 208	+ 6.0
Leverett Circle Ramps, Sta. 64 to 72	+ 10.0
Leverett Circle Ramps, Sta. 72 to 92	+ 6.0
South Boston	+ 7.0
East Boston	+ 7.0

Table 18

UNIT WEIGHTS FOR SOIL TYPES

SOIL TYPE	CONDITION	UNIT WEIGHT (PCF) MAX.	MIN.	REMARKS
Miscellaneous Fill (Existing)	Moist	120	100	
	Saturated	125	100	
	Submerged	62.5	36	
Silt, Fine Sandy Silt	Saturated	110	90	Typically Organic
	Submerged	47.5	26	
Clay	Saturated	120	110	Boston Blue Clay
	Submerged	57.5	46	
Glacial Till	Saturated	140	125	
	Submerged	77.5	61	
New Fill Placed Under Water w/o Compaction	Saturated	110	100	
	Submerged	47.5	36	
New Fill Placed Under Water With Surcharge or Compaction	Saturated	130	120	
	Submerged	67.5	56	
New Fill Placed Above Water w/o Compaction	Moist	110	90	
New Fill Placed Above Water With Compaction	Moist	130	110	

Table 19

COEFFICIENTS OF LATERAL EARTH PRESSURE

$$p_h = \bar{p}_h + u_o$$

$$\bar{p}_{h_o} = K_o \bar{p}_{v_o}$$

$$\bar{p}_{h_a} = K_a \bar{p}_{v_o}$$

K_a: Coefficient of Active Earth Pressure.

K_o: Coefficient of Earth Pressure at Rest.

SOIL TYPE	COEFFICIENT K_a	COEFFICIENT K_o	REMARKS
Miscellaneous Fill	0.33	0.45	
Silt, Fine Sandy Silt	0.30	0.45	Typically Organic
Clay (Boston Blue Clay)	0.35	0.55	
Glacial Till	0.27	0.45	
Placed Fill	0.30	0.45	

p_v, p_h: Total Vertical and Horizontal Earth Pressure.

$\bar{p}_h$: Effective Horizontal Earth Pressure.

$\bar{p}_{h_o}$: Effective Horizontal Earth Pressure at Rest.

$\bar{p}_{h_a}$: Effective Active Horizontal Earth Pressure.

Table 20

ALLOWABLE SOIL BEARING PRESSURES

SOIL TYPE	ALLOWABLE BEARING PRESSURE (TSF)	REMARKS
Clay	1 - 4	Boston Blue Clay Formations; -Will Vary With Site-Specific Conditions; Range Typical For Medium Stiff to Hard Clay
Glacial Till	4 - 10	
Granular Backfill	4	Structural Fill: Meets Specified Gradation and Compaction Criteria

NOTE:

Existing miscellaneous surface fills and the underlying organic silts are generally not suitable as foundation bearing materials.

Table 21

MODULUS OF SUBGRADE REACTION: k_v

(For a Rigid 1 Ft. x 1 Ft. Foundation Unit)

(lb. per sq. in. per in.)

SOIL TYPE	k_v: MODULUS OF SUBGRADE REACTION	R E M A R K S
Clay	50 - 100	
Glacial Till	200 - 300	
Granular Backfill	100 - 200	Structural Fill: Meets Specified Gradation and Compaction Criteria.

NOTE:

Values represent range considering variations in soil conditions. Other than soil type, modulus of subgrade reaction depends on surface disturbance due to excavation, width and shape of foundation interacting with the soil and other factors.

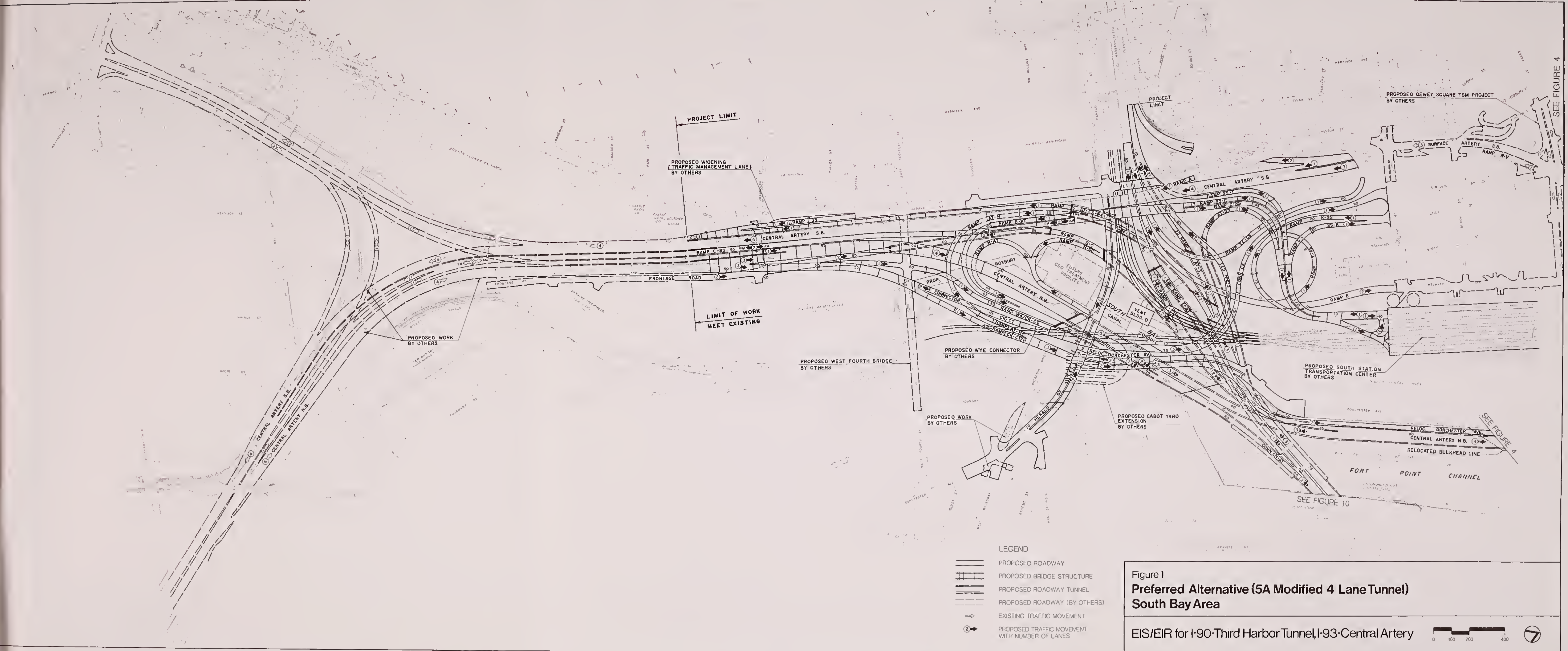


Figure 1
Preferred Alternative (5A Modified 4 Lane Tunnel)
South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

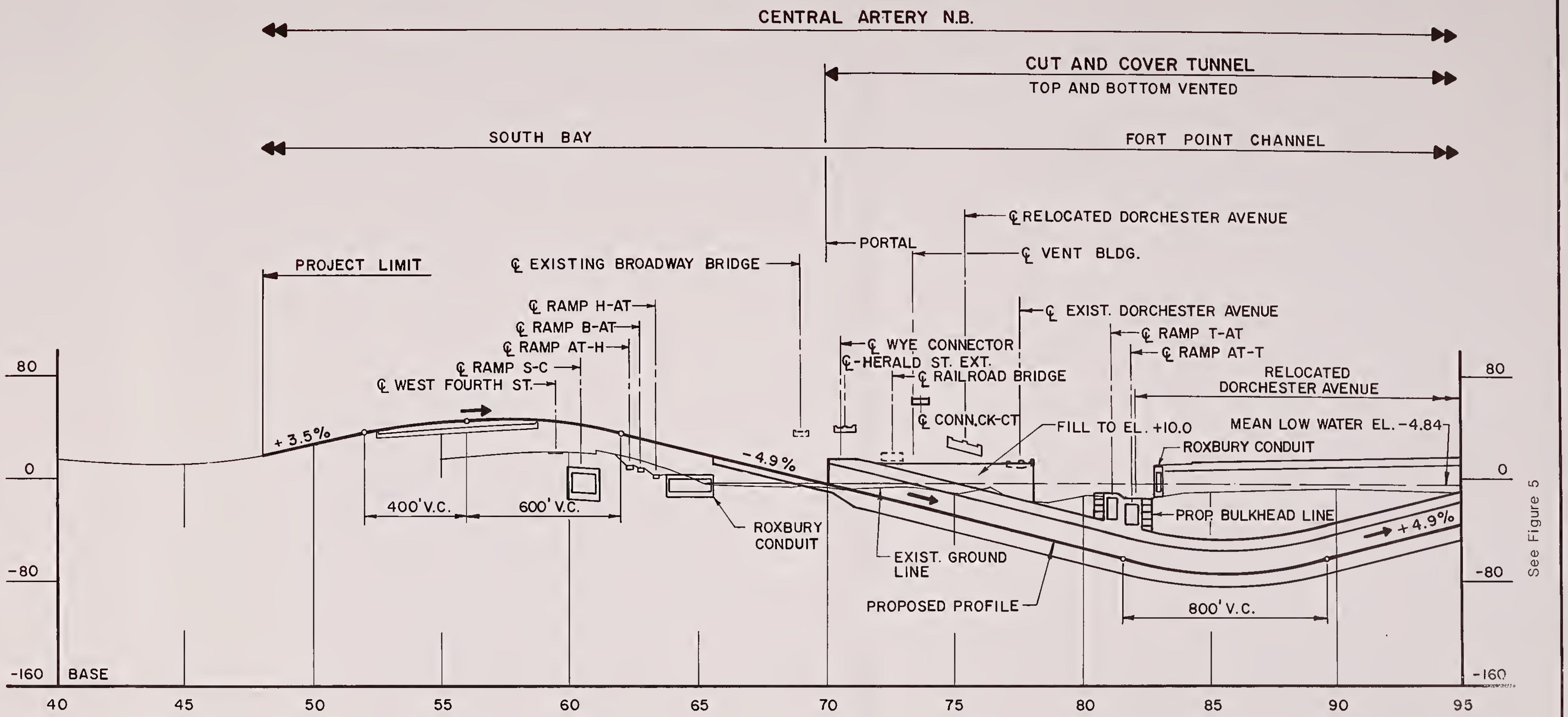


Figure 2
Preferred Alternative (5A Modified 4 Lane Tunnel)
South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Horizontal Scale 0 200 400 Feet
Vertical Scale 0 40 80

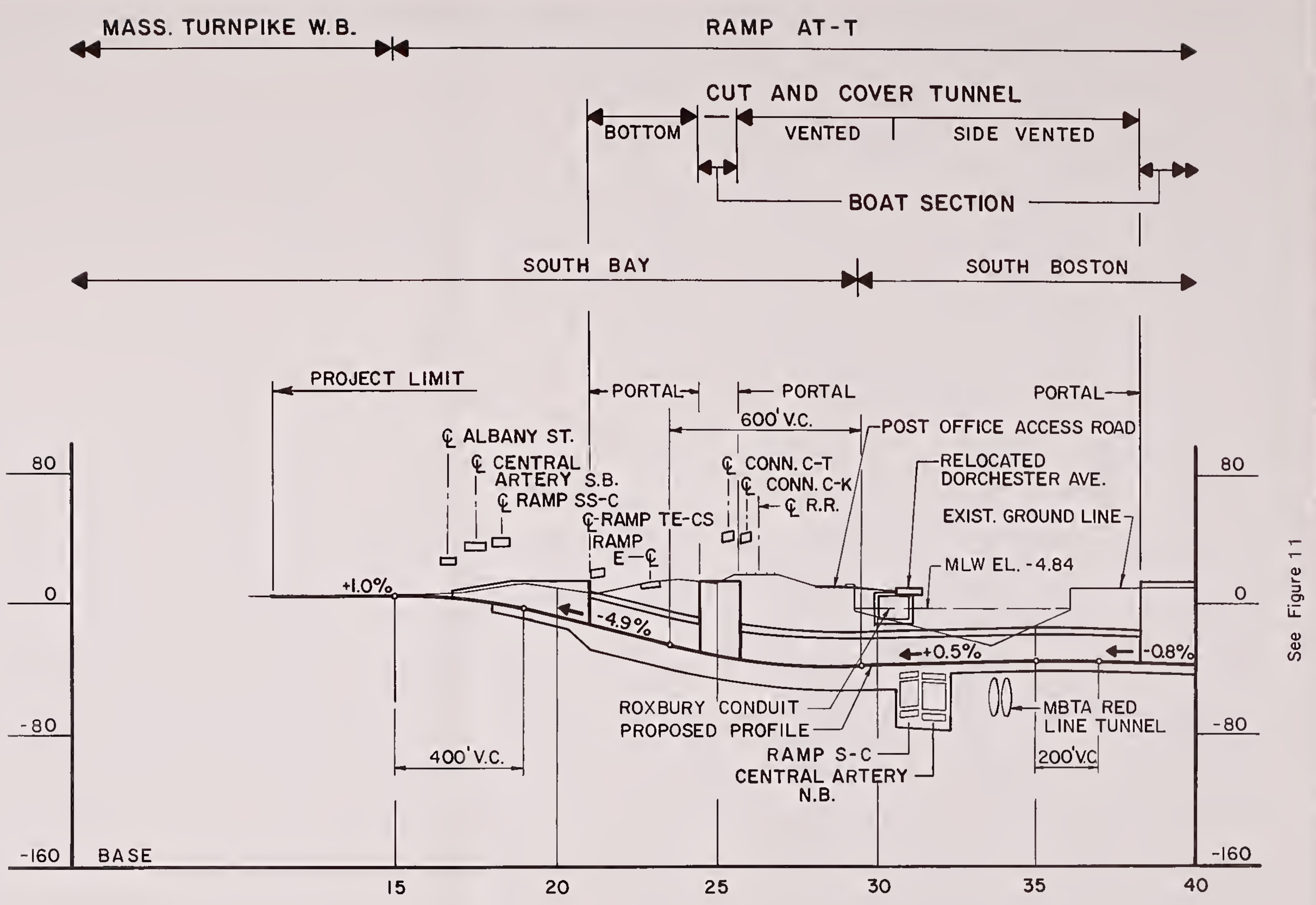


Figure 3
Preferred Alternative (5A Modified 4 Lane Tunnel)
South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Horizontal Scale 0 200 400 Feet
Vertical Scale 0 40 80

Figure 4
Preferred Alternative (5A Modified 4 Lane Tunnel)
Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

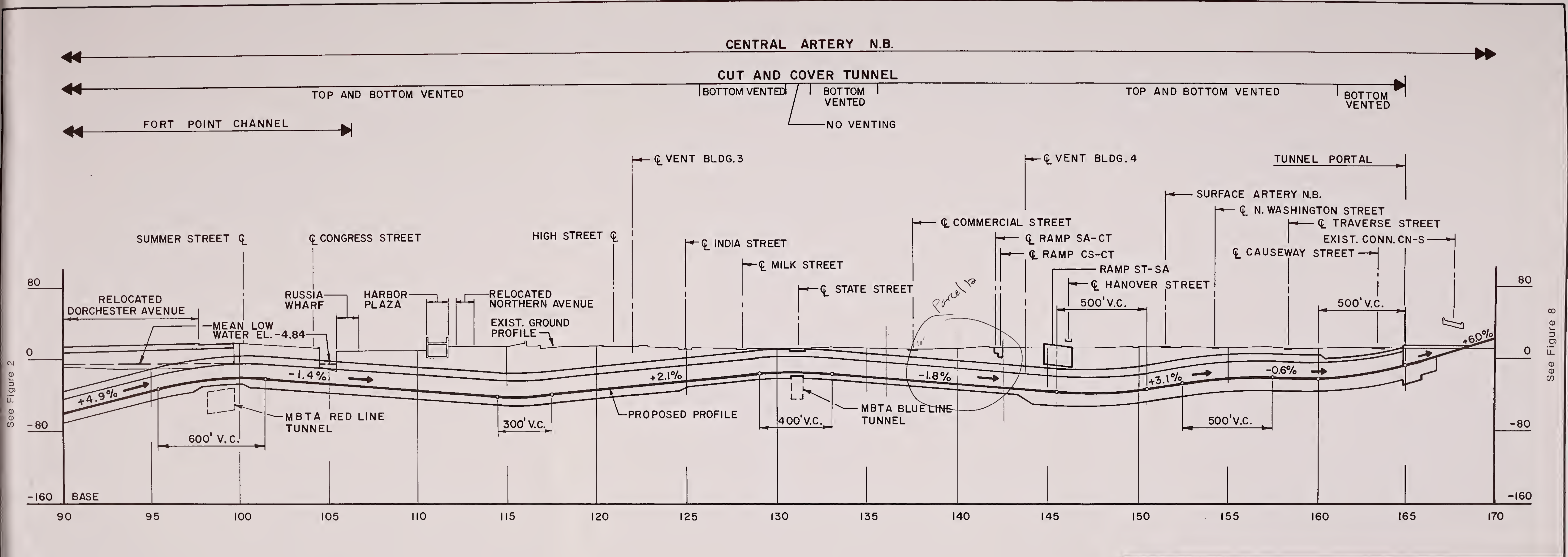


Figure 5

Preferred Alternative (5A Modified 4 Lane Tunnel) Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Horizontal Scale 0 200 400 Feet
Vertical Scale 0 40 80

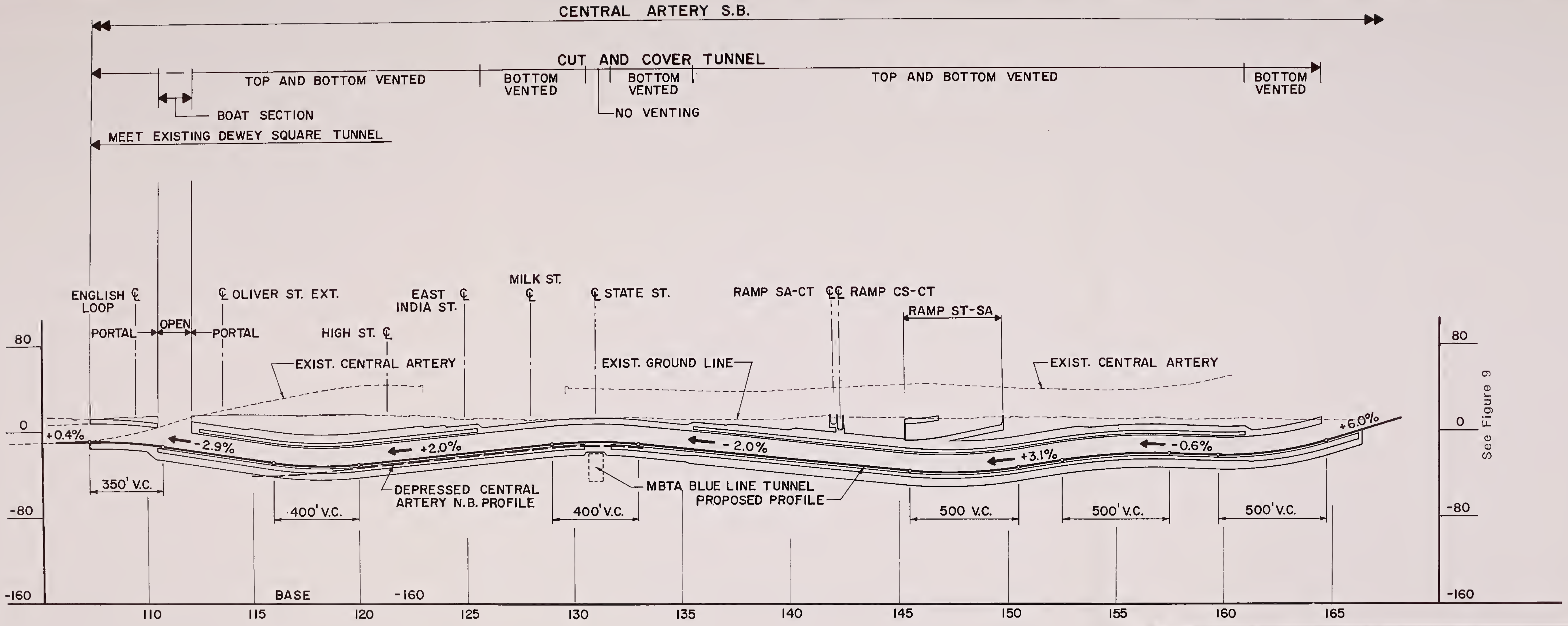


Figure 6

Preferred Alternative (5A Modified 4 Lane Tunnel)
Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Horizontal Scale 0 200 400 Feet
Vertical Scale 0 40 80

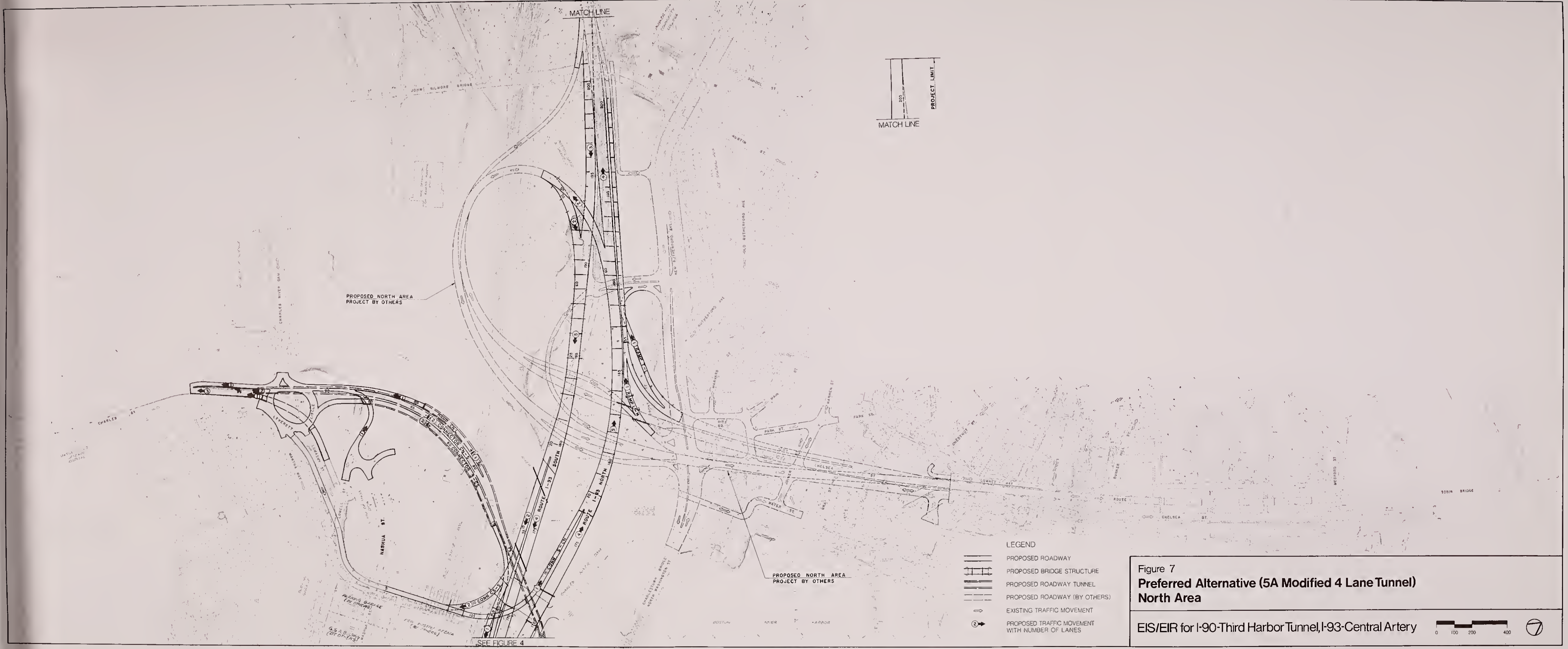


Figure 7
Preferred Alternative (5A Modified 4 Lane Tunnel) North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

See Figure 5

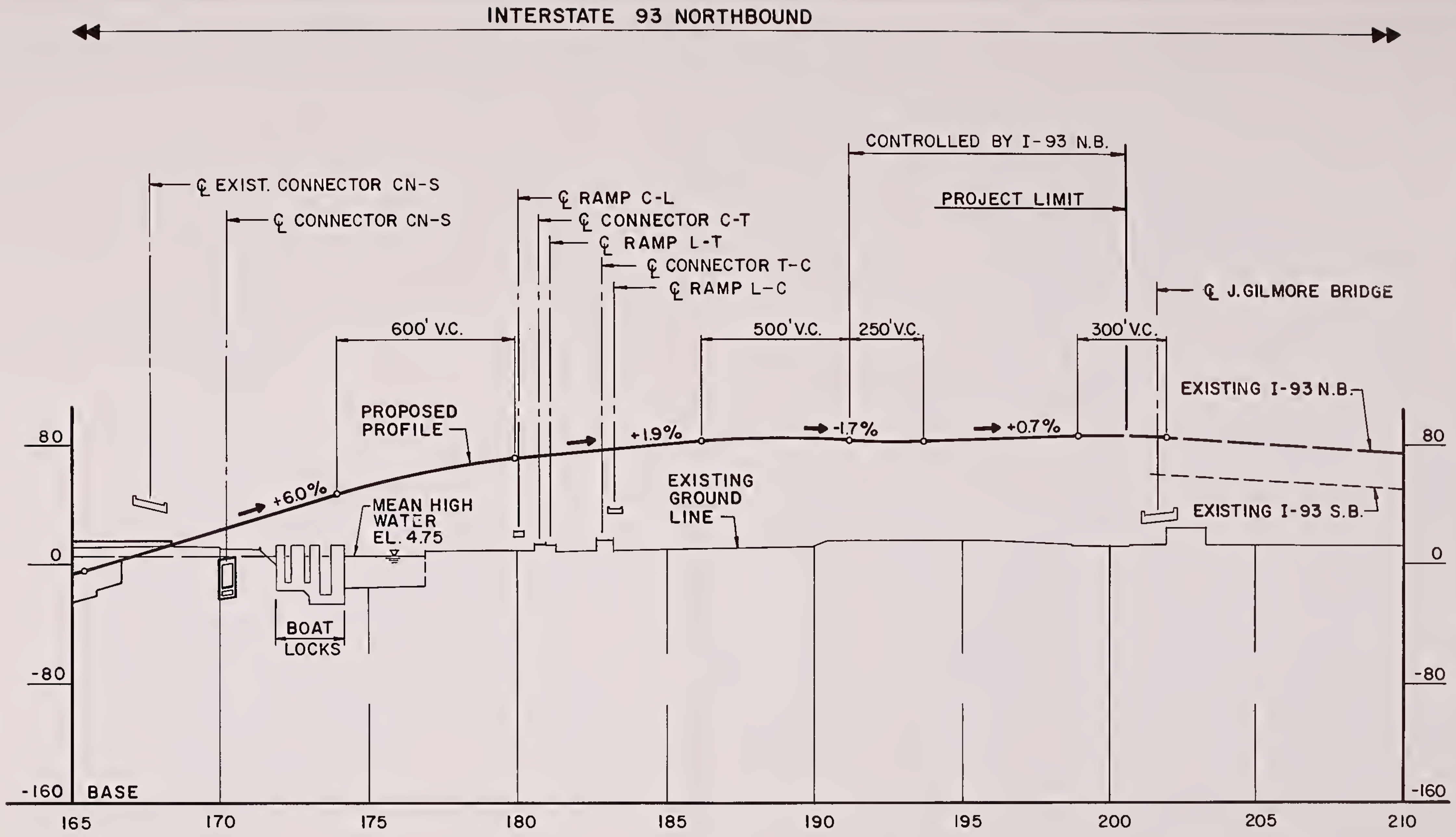


Figure 8
Preferred Alternative (5A Modified 4 Lane Tunnel)
North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Horizontal Scale 0 200 400 Feet
Vertical Scale 0 40 80

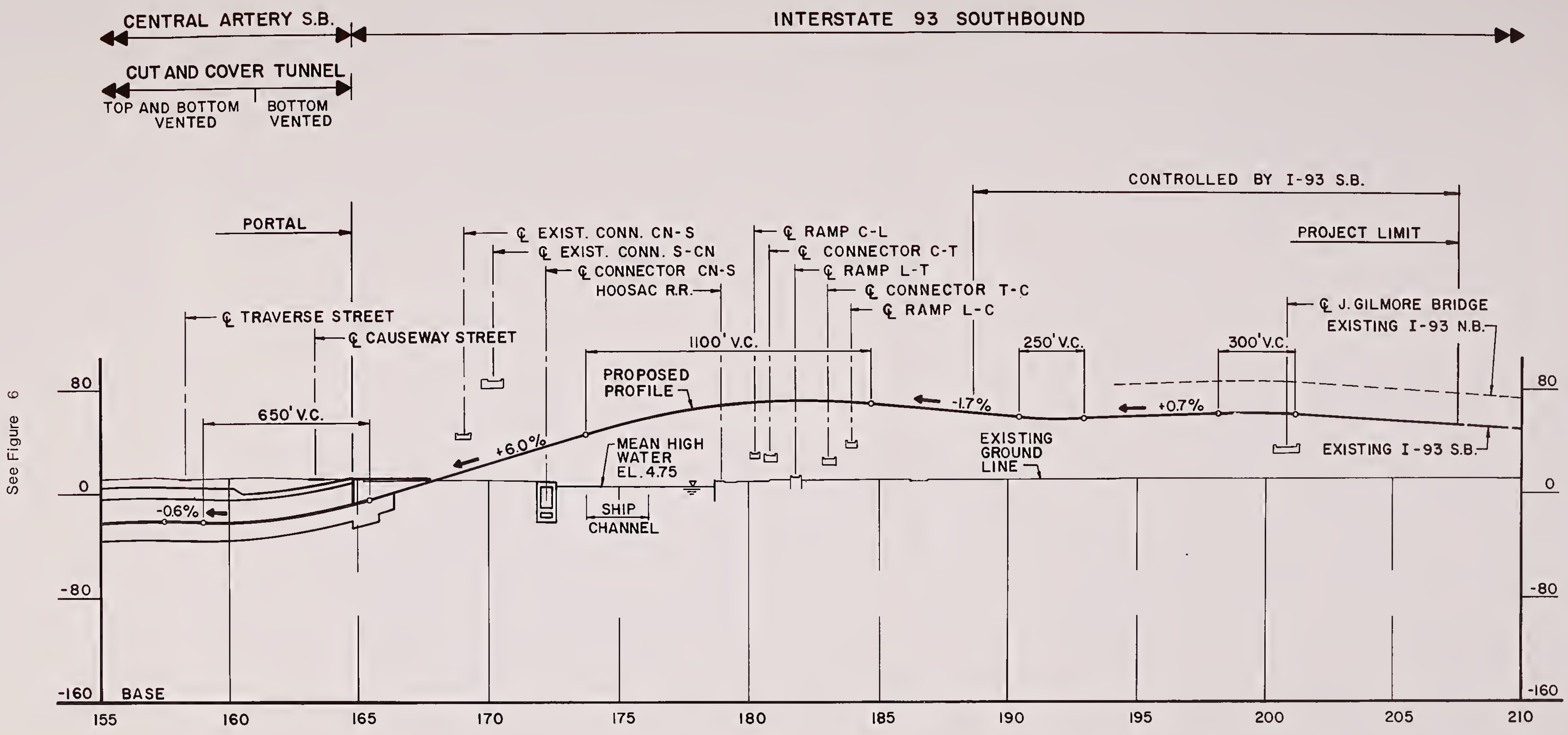


Figure 9
Preferred Alternative (5A Modified 4 Lane Tunnel)
North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Horizontal Scale 0 200 400 Feet
Vertical Scale 0 40 80

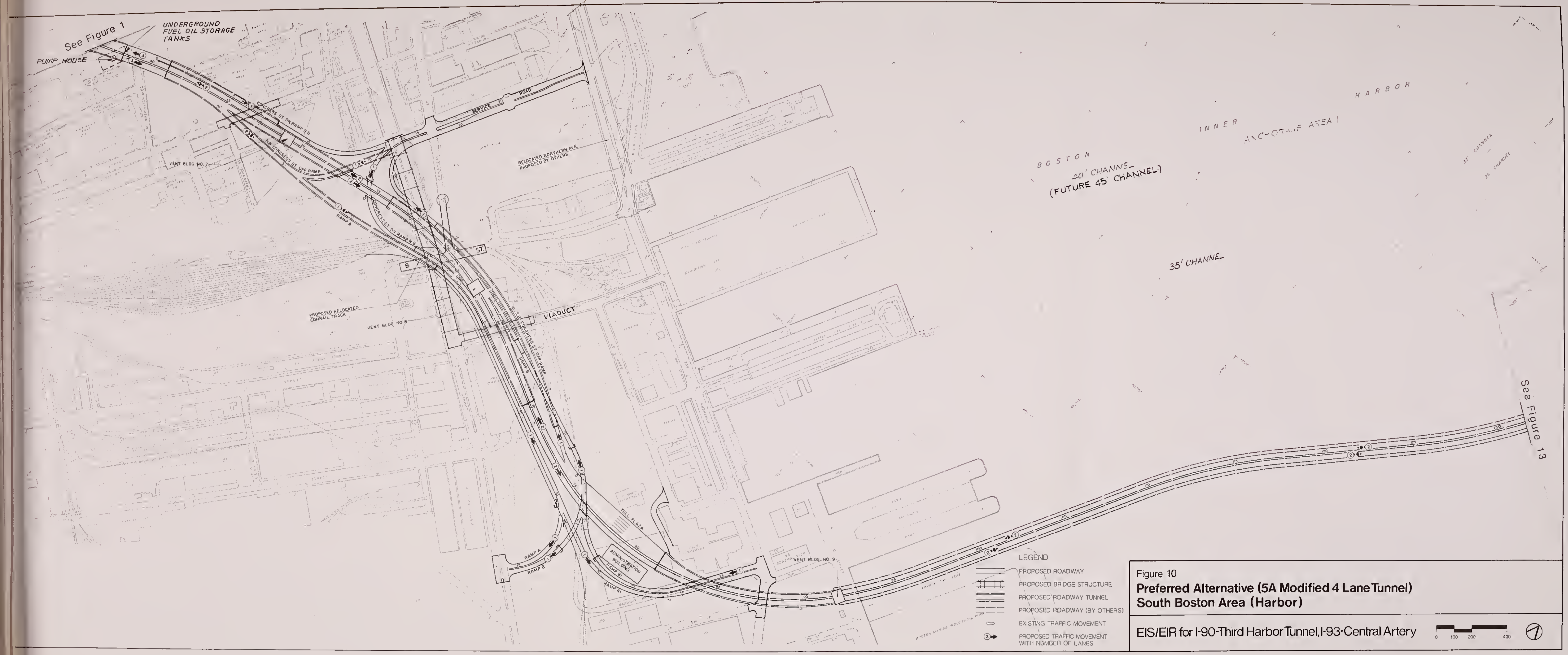


Figure 10
Preferred Alternative (5A Modified 4 Lane Tunnel)
South Boston Area (Harbor)

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

See Figure 3

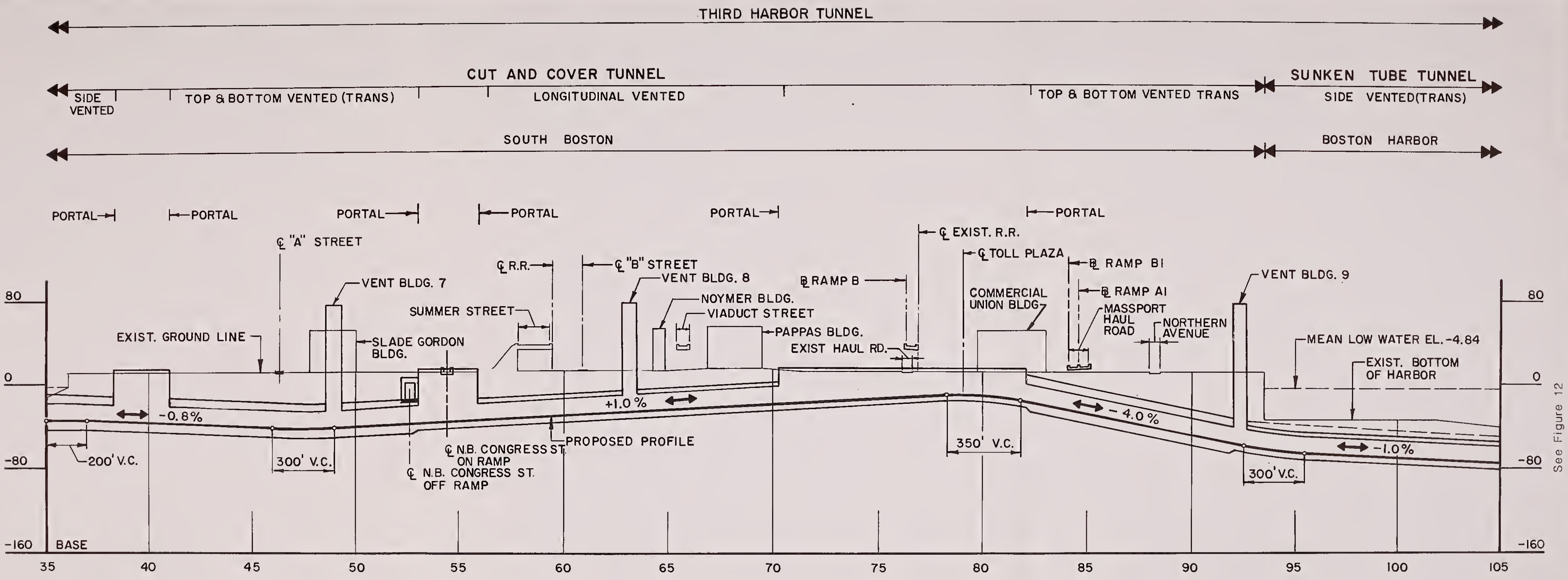


Figure 11
Preferred Alternative (5A Modified 4 Lane Tunnel)
South Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Horizontal Scale 0 200 400 Feet
Vertical Scale 0 40 80

See Figure 12

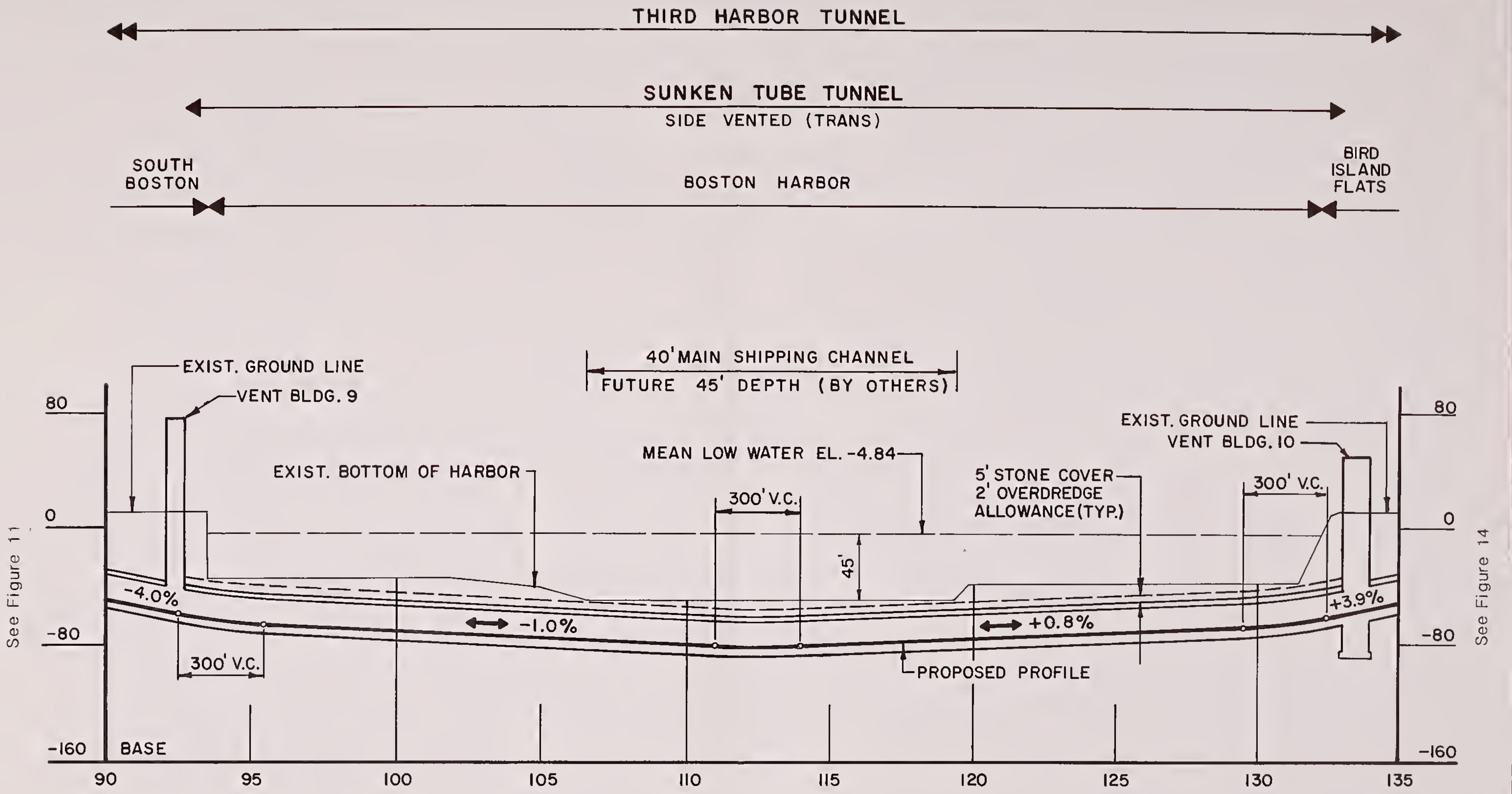


Figure 12
Preferred Alternative (5A Modified 4 Lane Tunnel)
South Boston Area (Harbor)

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Horizontal Scale 0 200 400 Feet
Vertical Scale 0 40 80

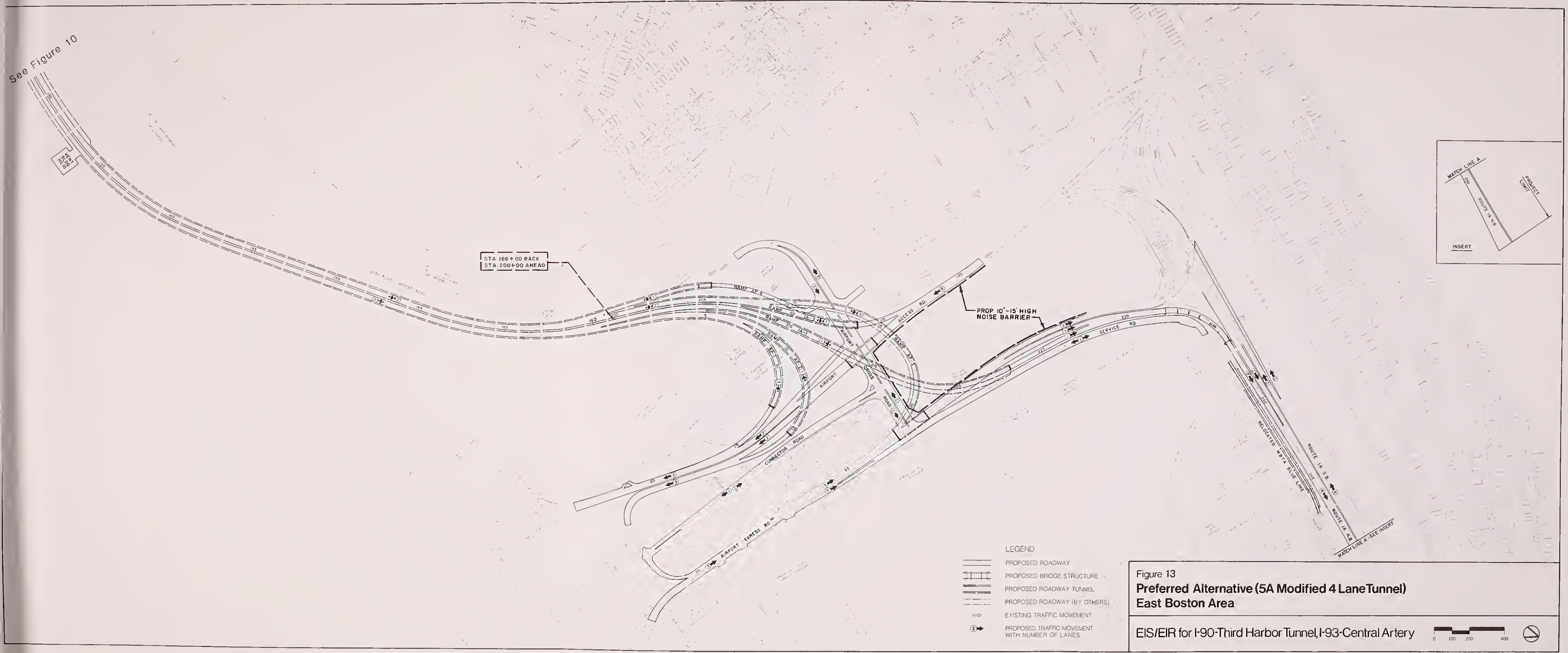


Figure 13
Preferred Alternative (5A Modified 4 Lane Tunnel)
East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

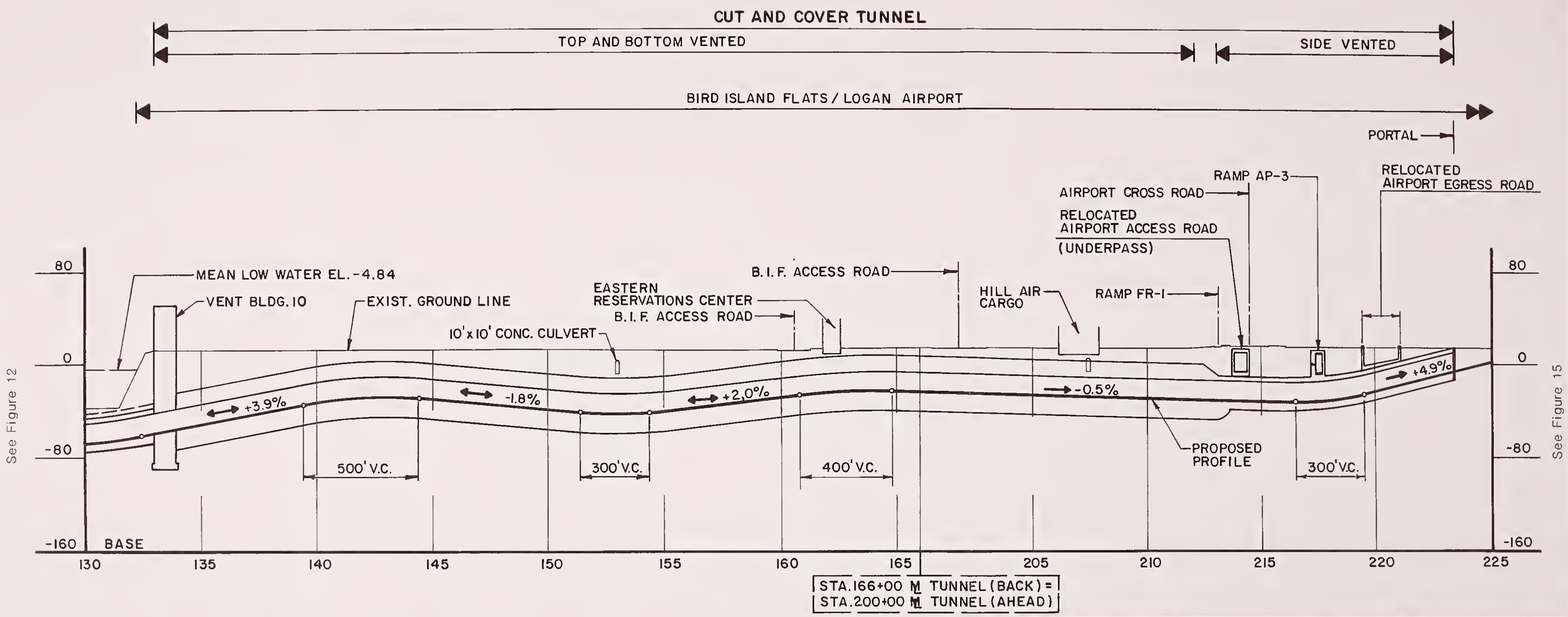
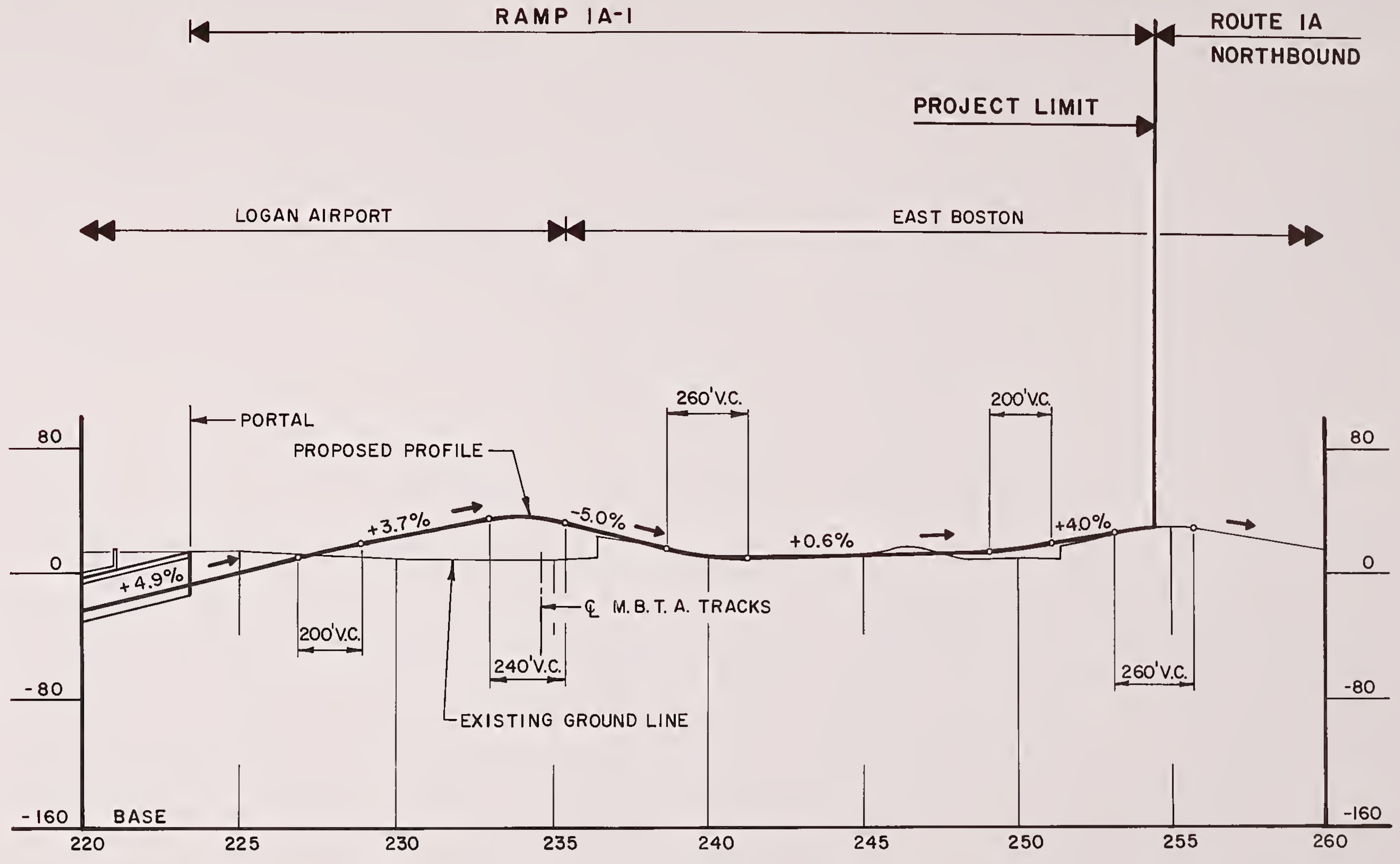


Figure 14
Preferred Alternative (5A Modified 4 Lane Tunnel)
East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Horizontal Scale 0 200 400 Feet
Vertical Scale 0 40 80

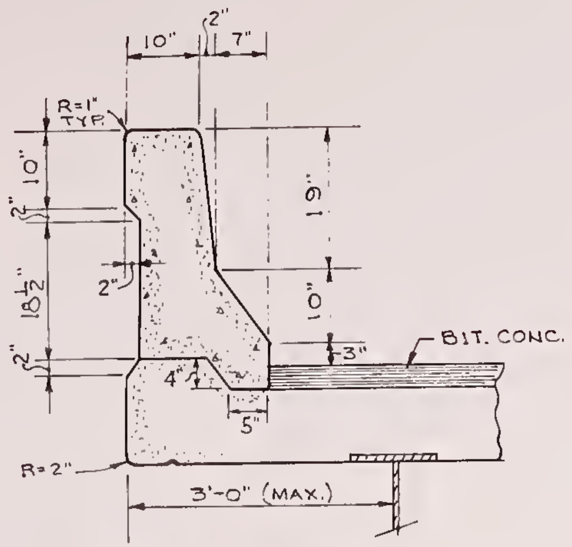


See Figure 14

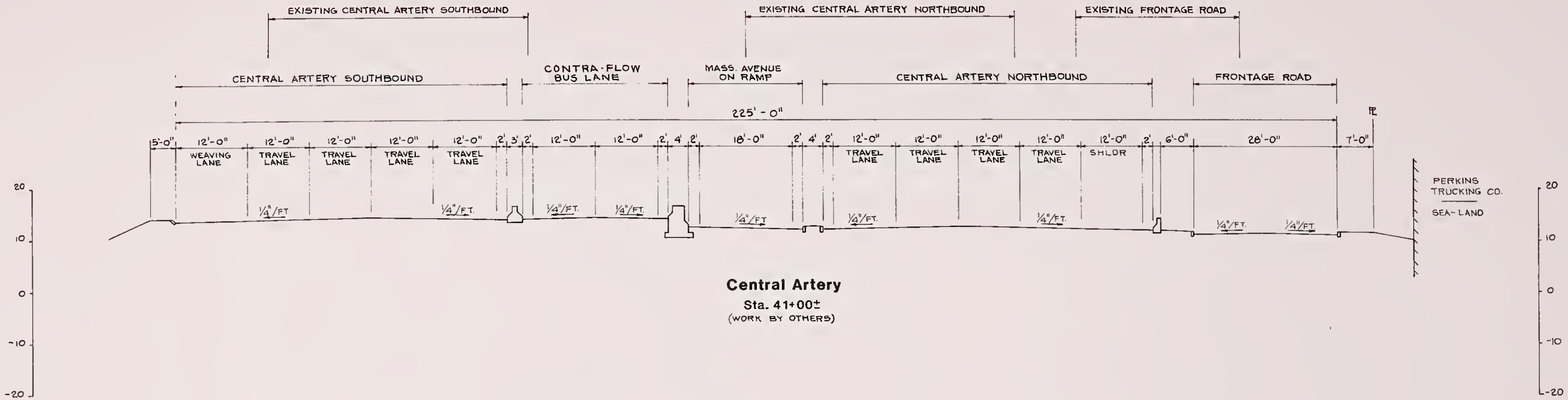
Figure 15
Preferred Alternative (5A Modified 4 Lane Tunnel)
East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Horizontal Scale 0 200 400 Feet
Vertical Scale 0 40 80



New Jersey Barrier (Bridge)



Central Artery
Sta. 41+00±
(WORK BY OTHERS)

GENERAL NOTES:
1. REFER TO FIGURE 25 FOR TUNNEL CURB OPTION DETAIL "A" & "B" (SAFETY WALK AND JERSEY BARRIER)

Figure 16
Preferred Alternative (5A Modified 4 Lane Tunnel)-Typical Sections South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet

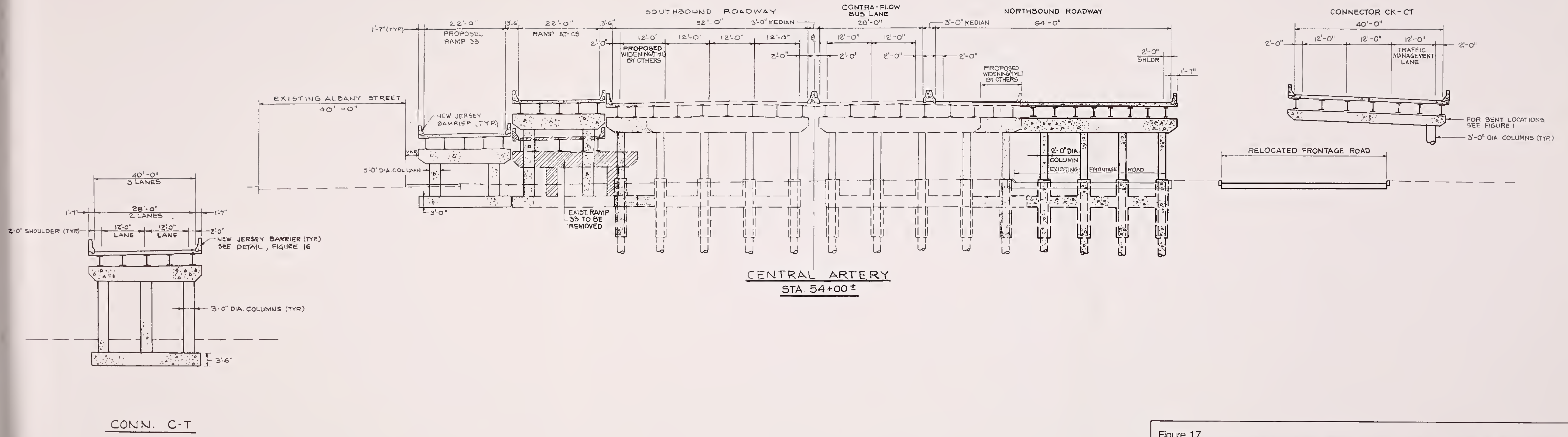


Figure 17

Preferred Alternative (5A Modified 4 Lane Tunnel)–Typical Sections South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet

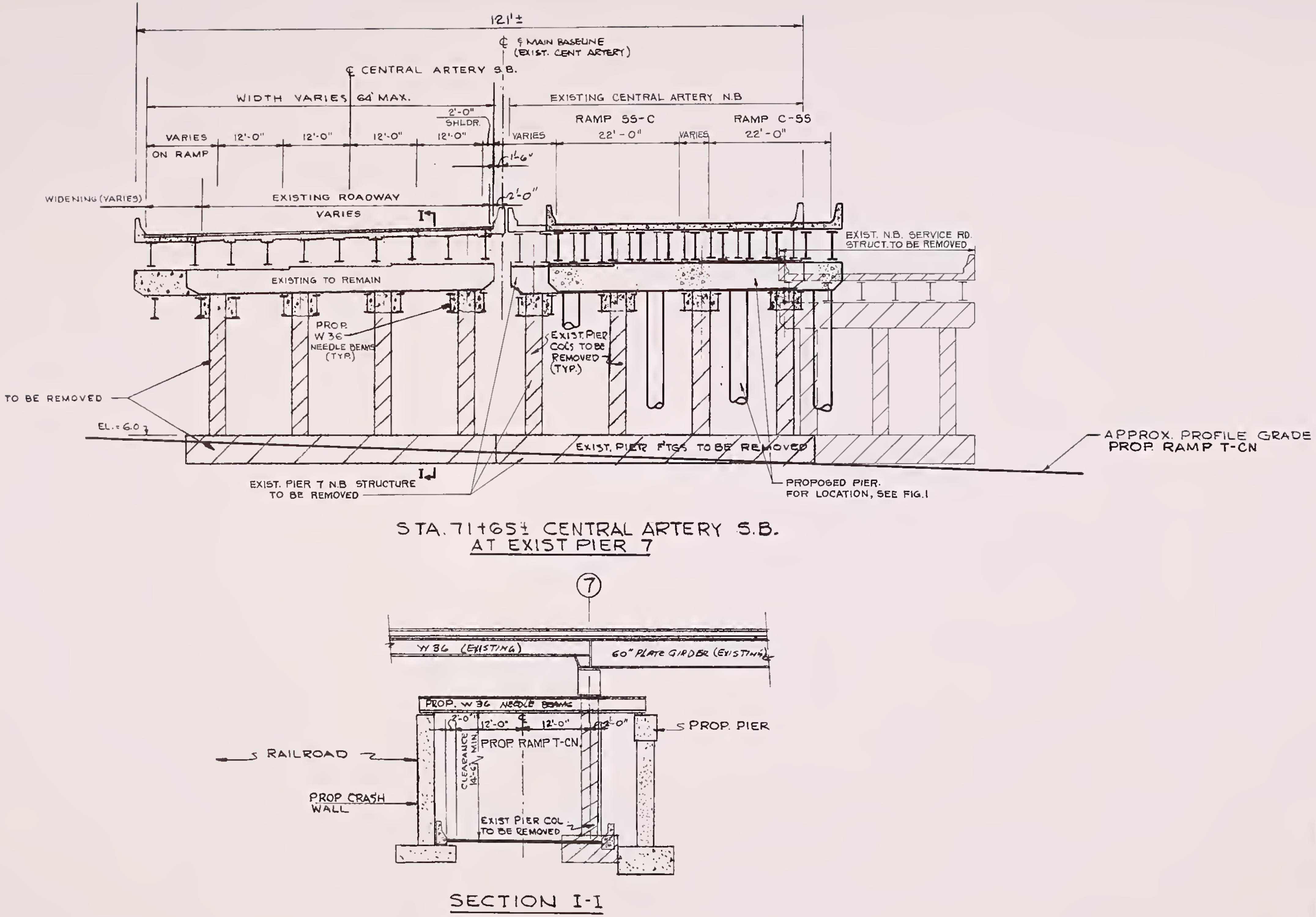


Figure 18
Preferred Alternative (5A Modified 4 Lane Tunnel)–Typical Sections South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

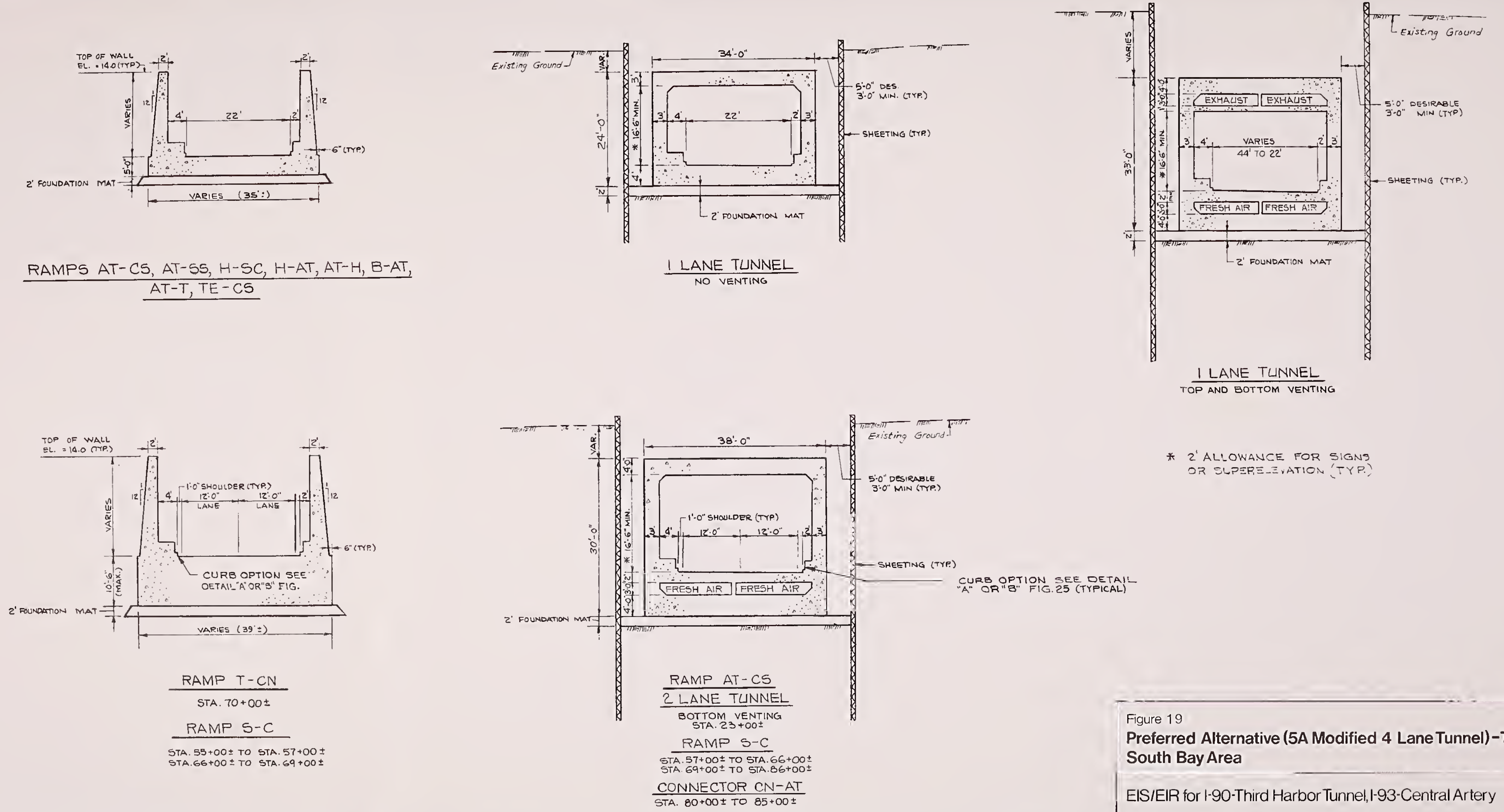
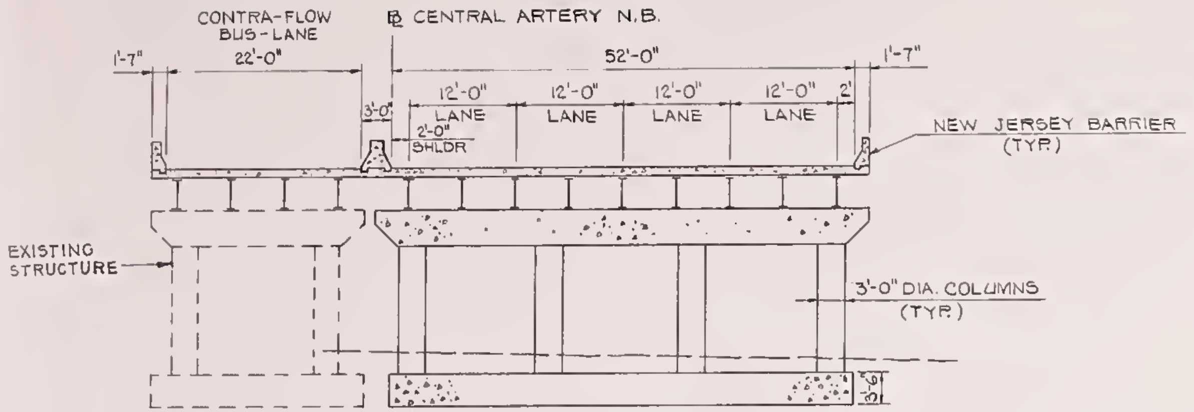
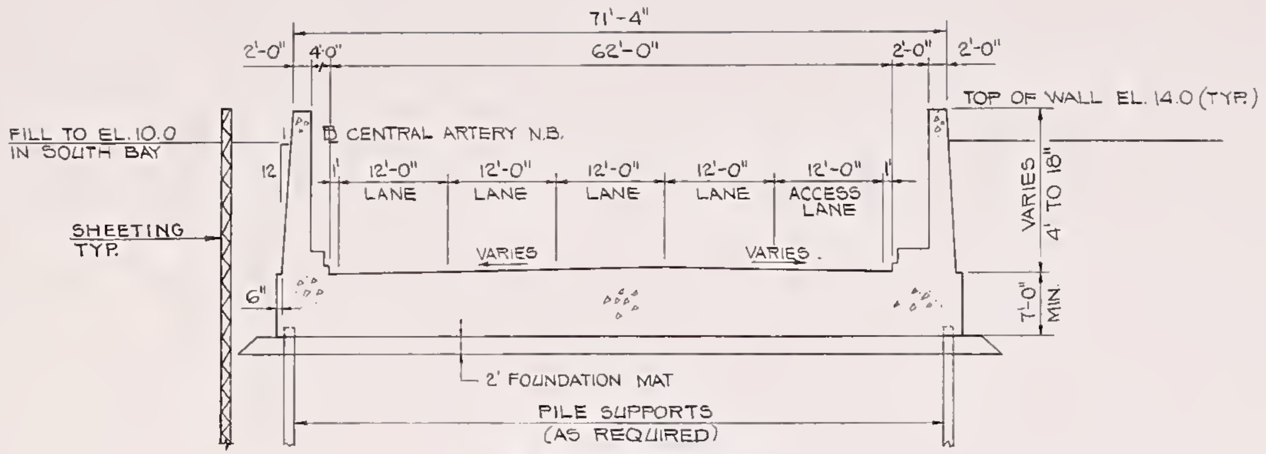


Figure 19
Preferred Alternative (5A Modified 4 Lane Tunnel) – Typical Sections South Bay Area

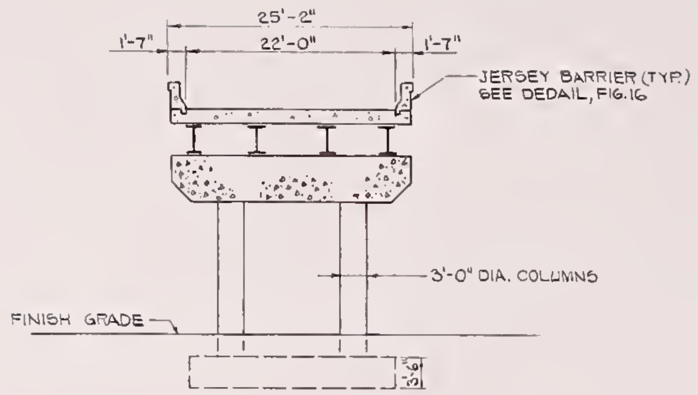
EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



Central Artery Northbound
Sta. 55-00±-Sta. 60-00±

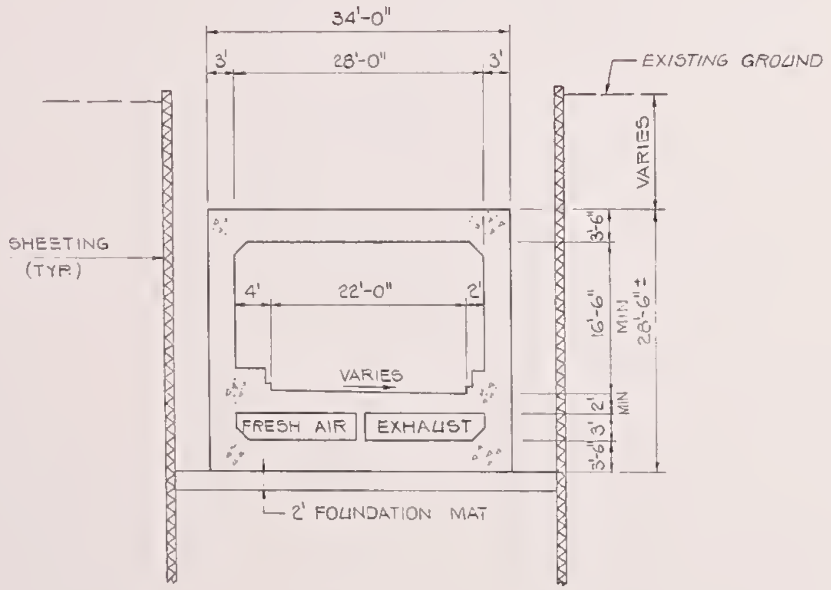


Central Artery Northbound
Sta. 67 + 00± - Sta. 70 + 00±



Ramps W4/CK-CT, AT-H, K-SS, SS-K, CONN. CK

Ramps C-SS, SS-C, SS-T, AT-SS, H-AT, AT-CS



Ramps AT-CS, T-CN, T-AT, AT-T, AT-SS, AT-H, H-AT

NOTES:

1. VERTICAL CLEARANCE 16'-6" PROVIDES 2' ALLOWANCE FOR SIGNS OR SUPERELEVATION. (TYPICAL)
2. SEE FIGURE 16 FOR TYPICAL N.J. BARRIER AND FIGURE 25 FOR CURB OPTIONS.

Figure 20
Preferred Alternative (5A Modified 4 Lane Tunnel) – Typical Sections South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet

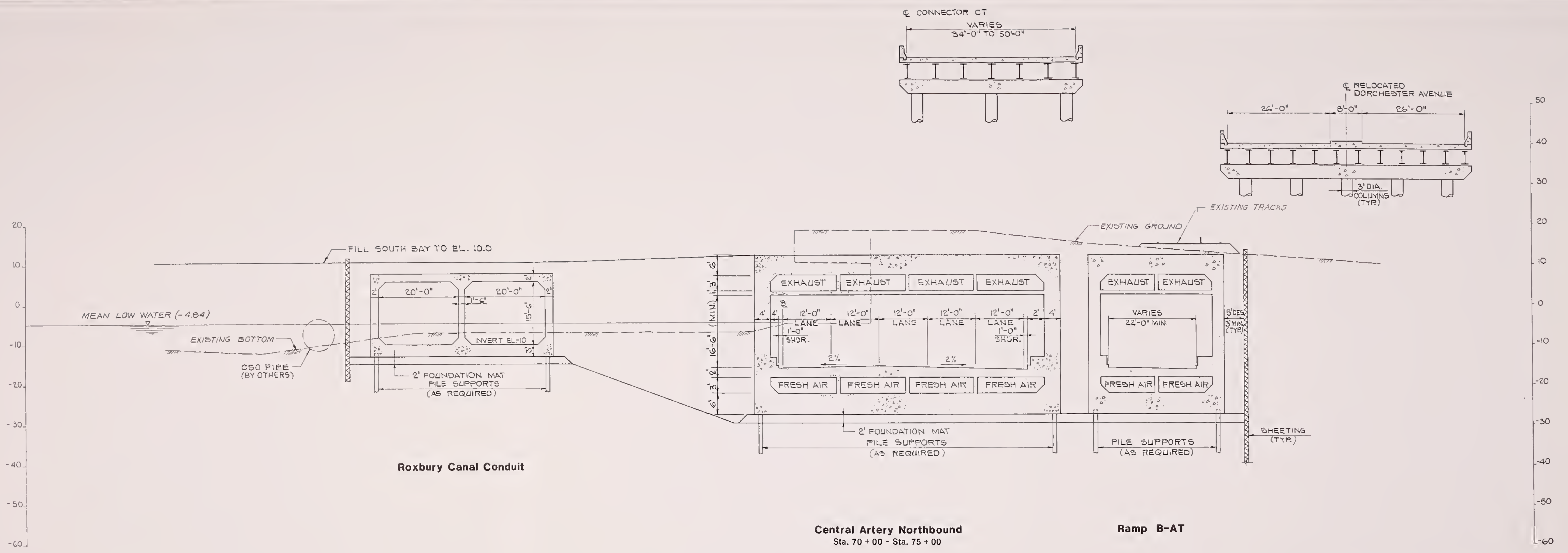


Figure 21
Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

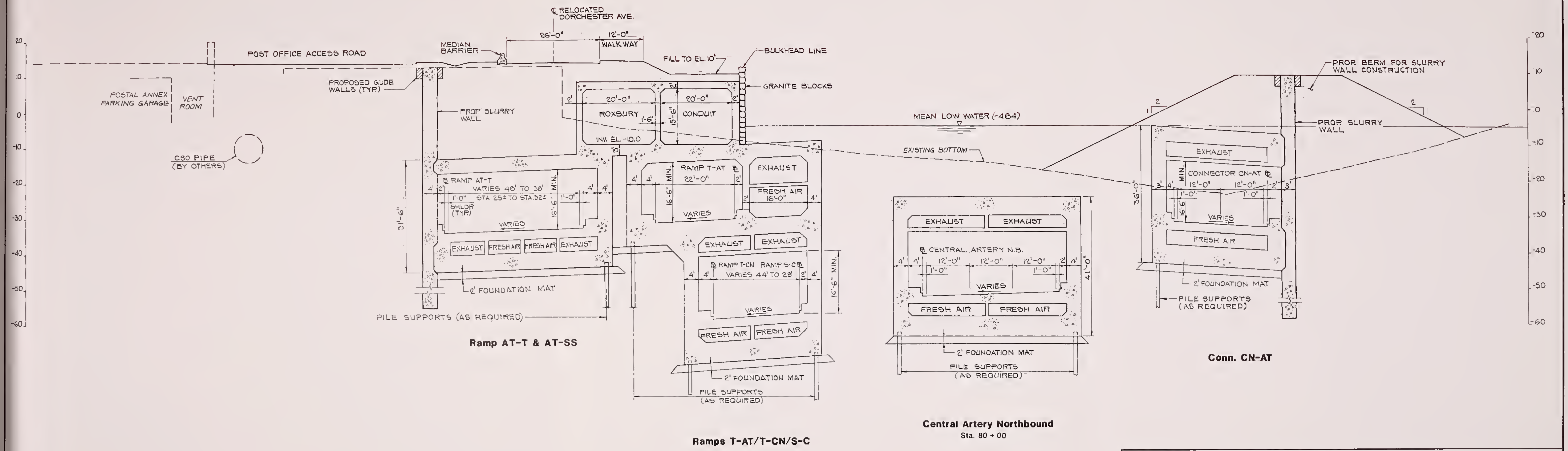
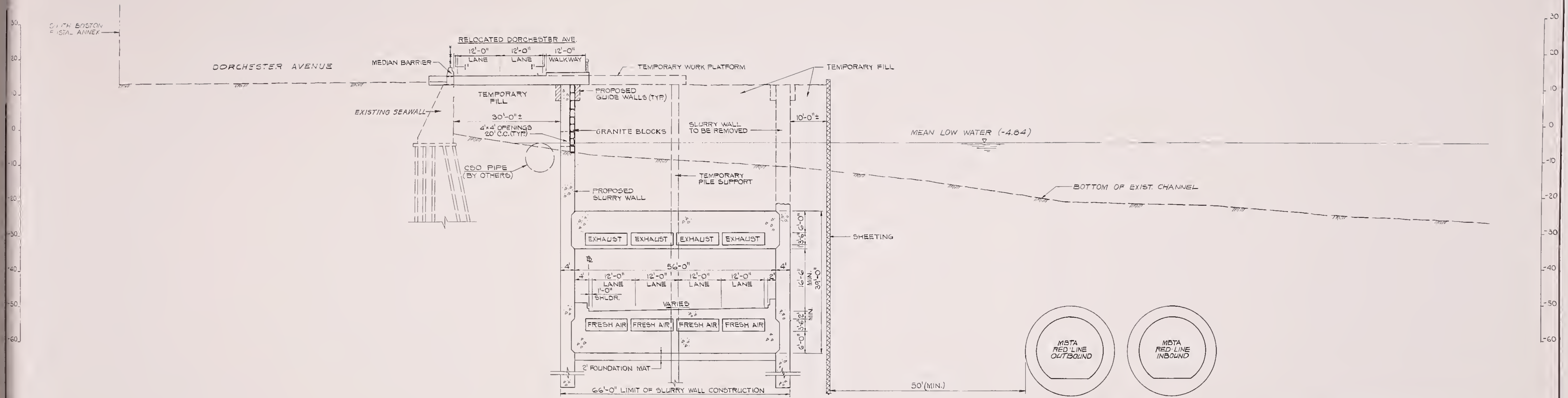


Figure 22

Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections South Bay Area

EIS/EIR for. I-90-Third Harbor Tunnel, I-93-Central Artery

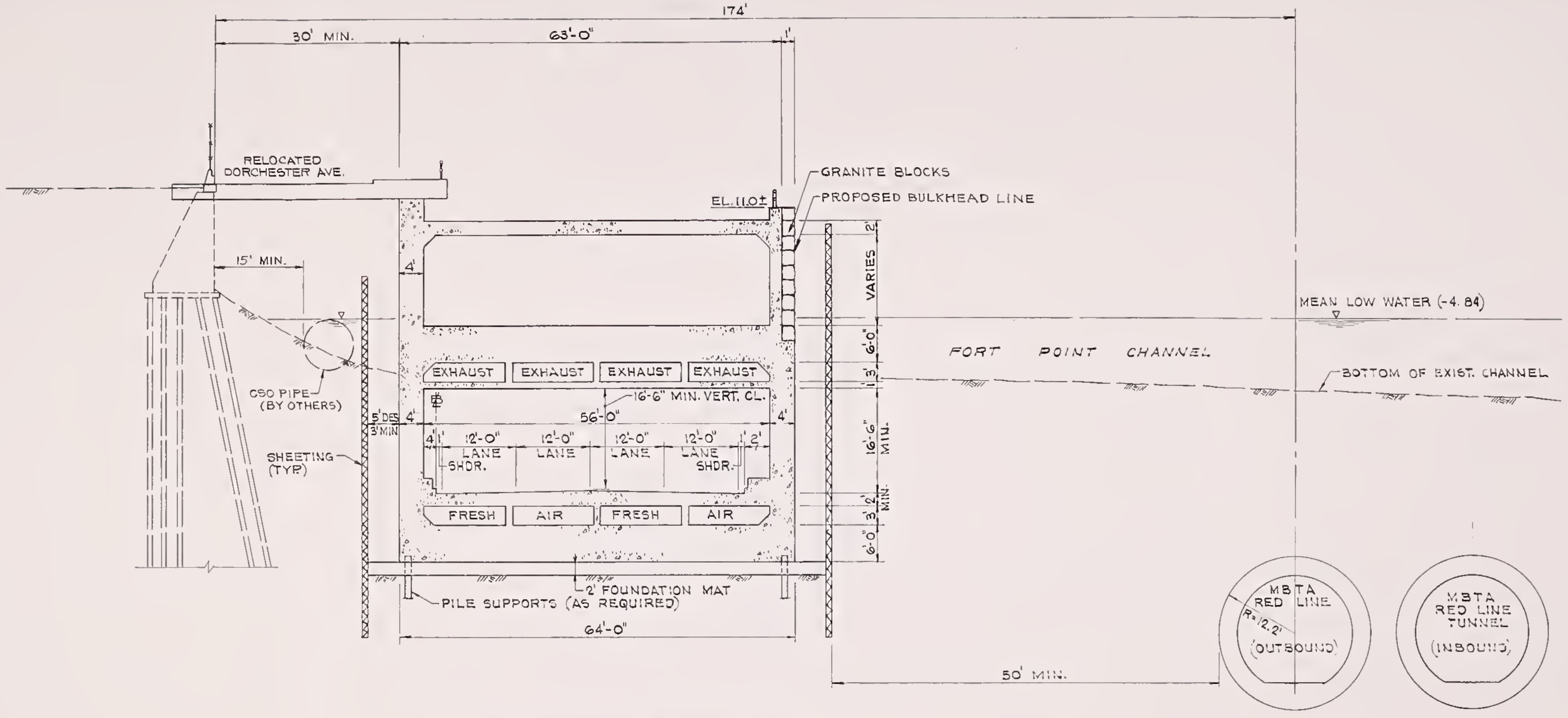


Central Artery Northbound
Sta. 83 + 00± - Sta. 94 + 00±

Figure 23
Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet

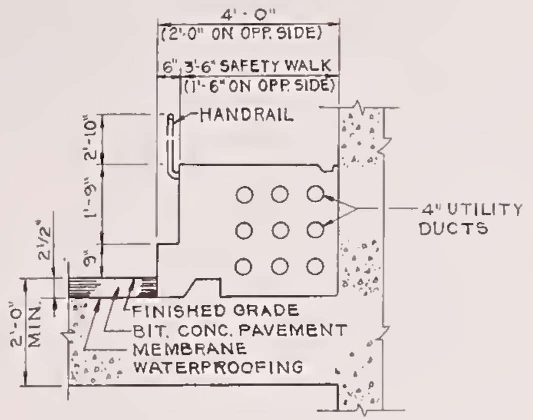


Central Artery Northbound - Fort Point Channel
Sta. 94+00 + 98-00

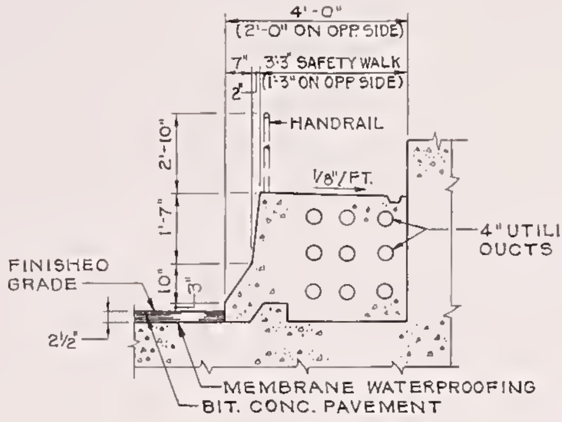
Figure 24
Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

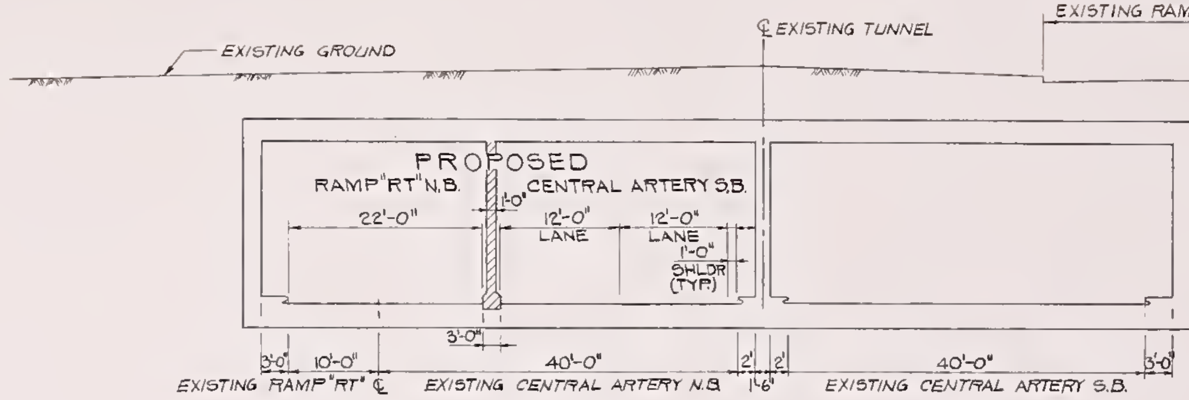
0 5 10 20 Feet



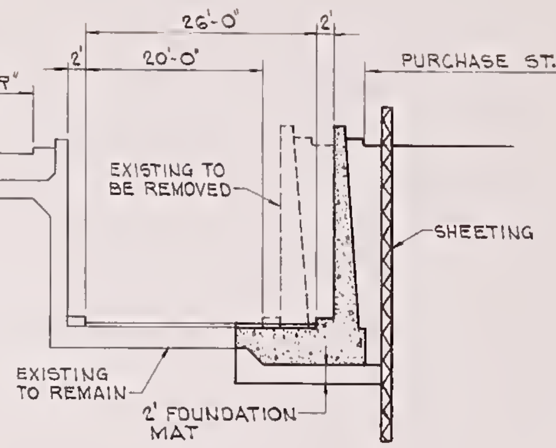
Detail "A"



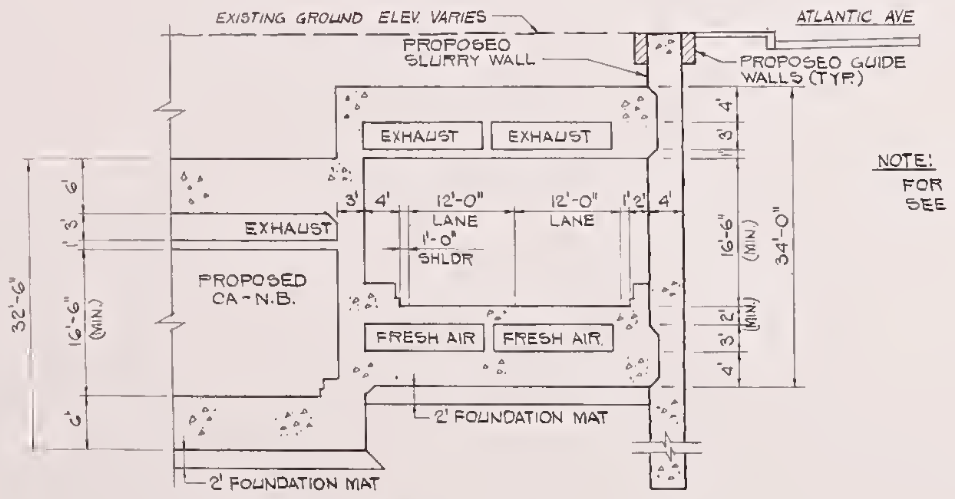
Detail "B"



Existing Dewey Square Tunnel At Proposed Sta. 102+00±
(Downstation)

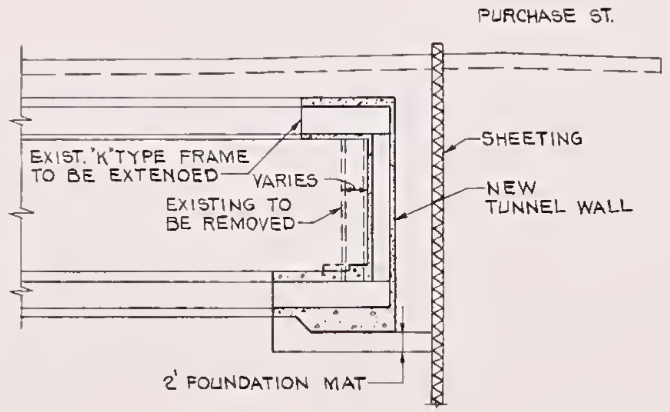


Ramp RS Widening
CONGRESS STREET



NOTE:
FOR 2-LANE BOAT SECTIONS, SEE FIG. 19

Ramp A-CN

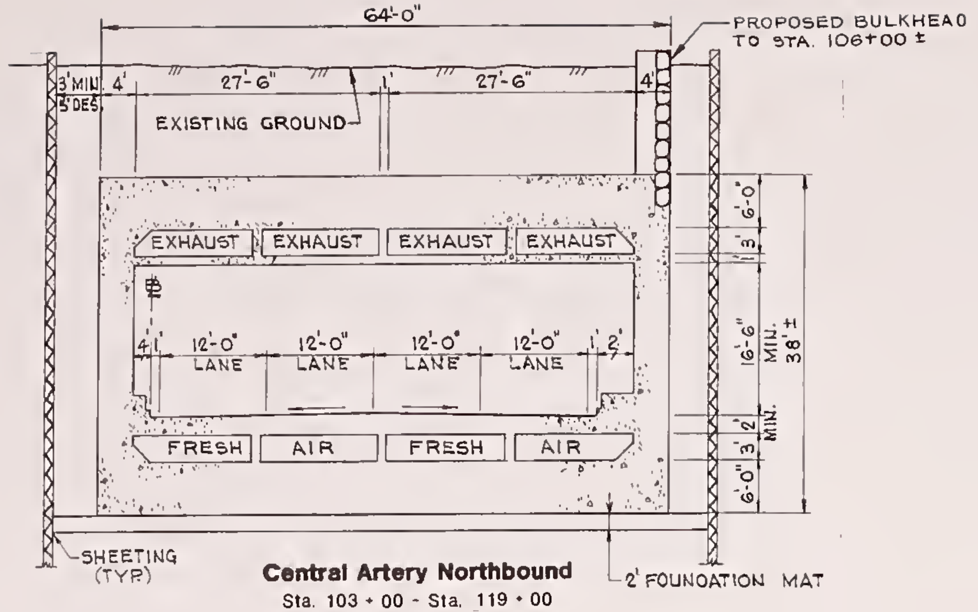


Tunnel Widening At Ramp RS
CONGRESS STREET

Figure 25
Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections Central Area

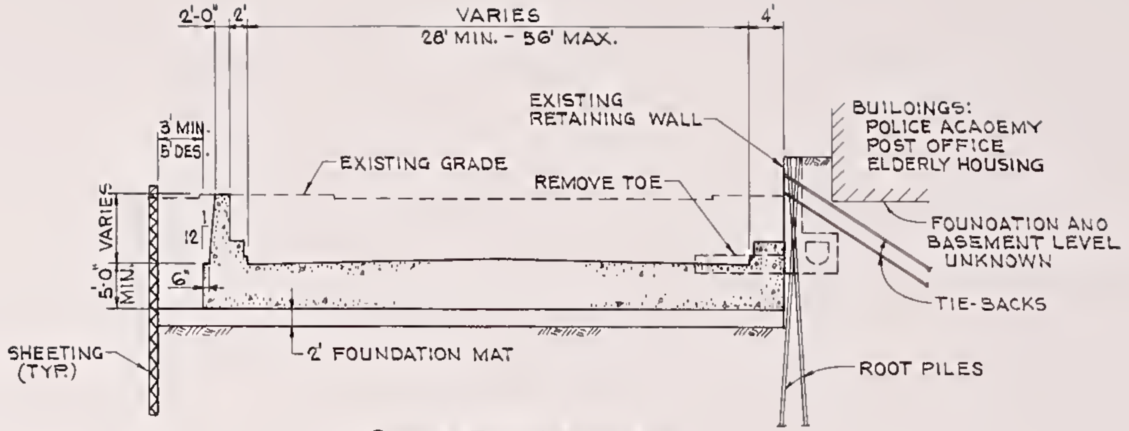
EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet

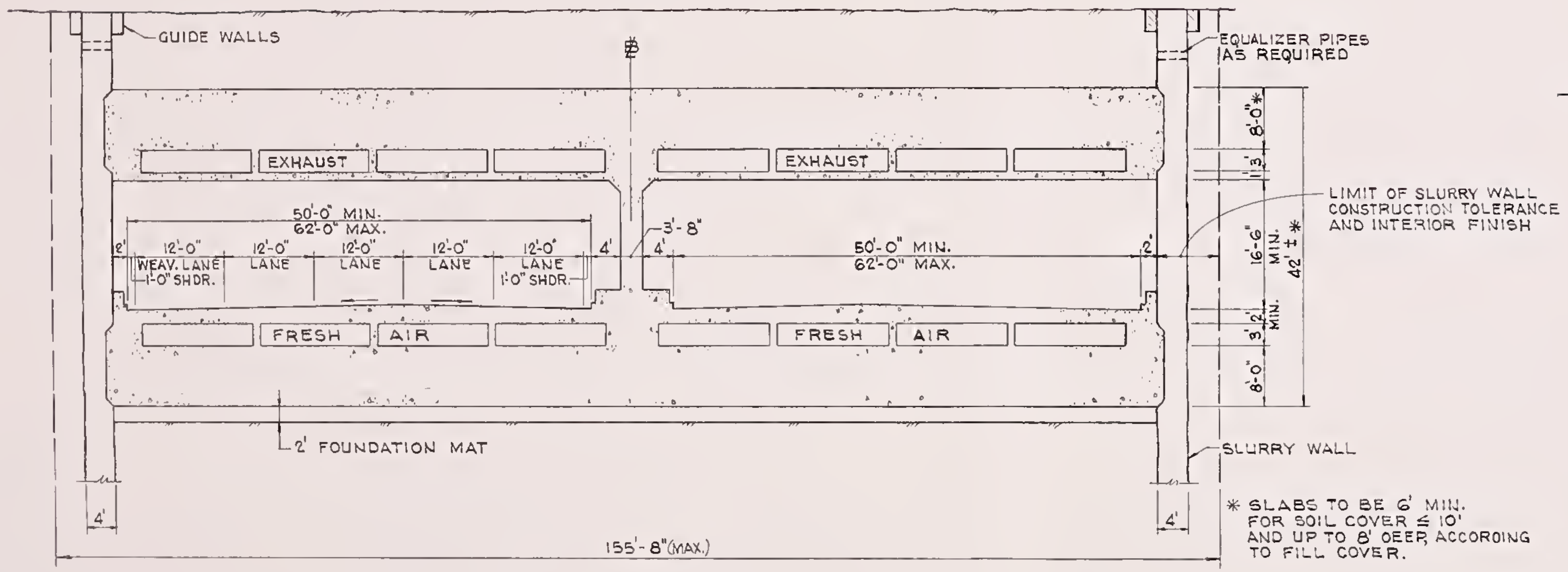


Central Artery Northbound

Sta. 103 + 00 - Sta. 119 + 00



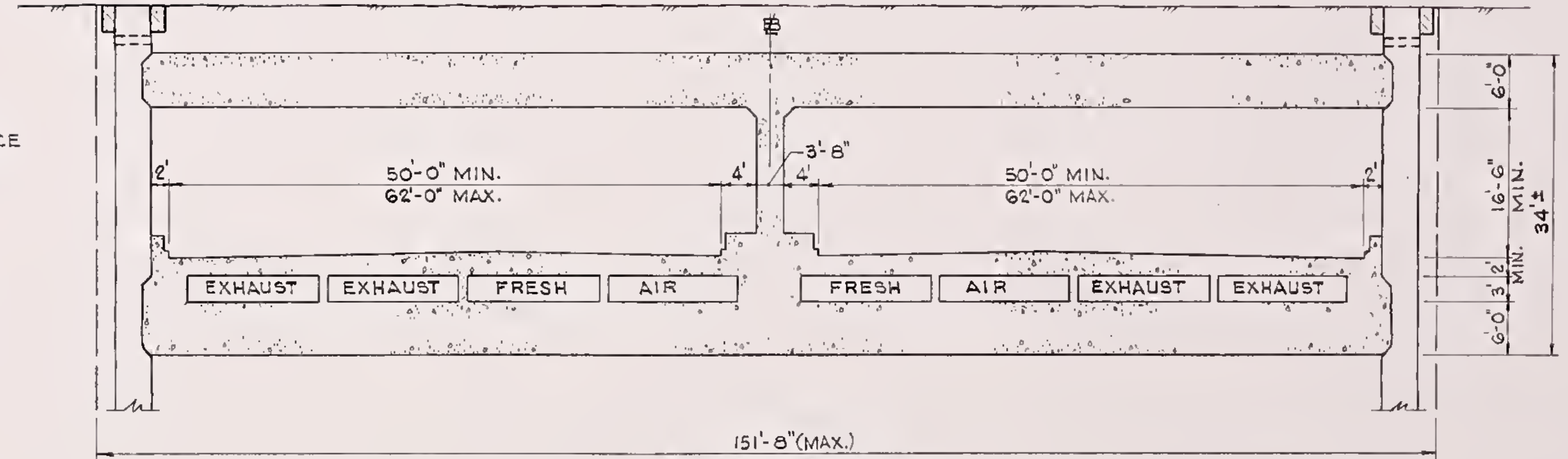
Sumner Tunnel Approach



Depressed Central Artery

Sta. 119 + 00 - Sta. 125 + 50

Sta. 135 + 50 - Sta. 160 + 00



Depressed Central Artery

Sta. 125 + 50 - Sta. 130 + 50

Sta. 131 + 70 - Sta. 135 + 50

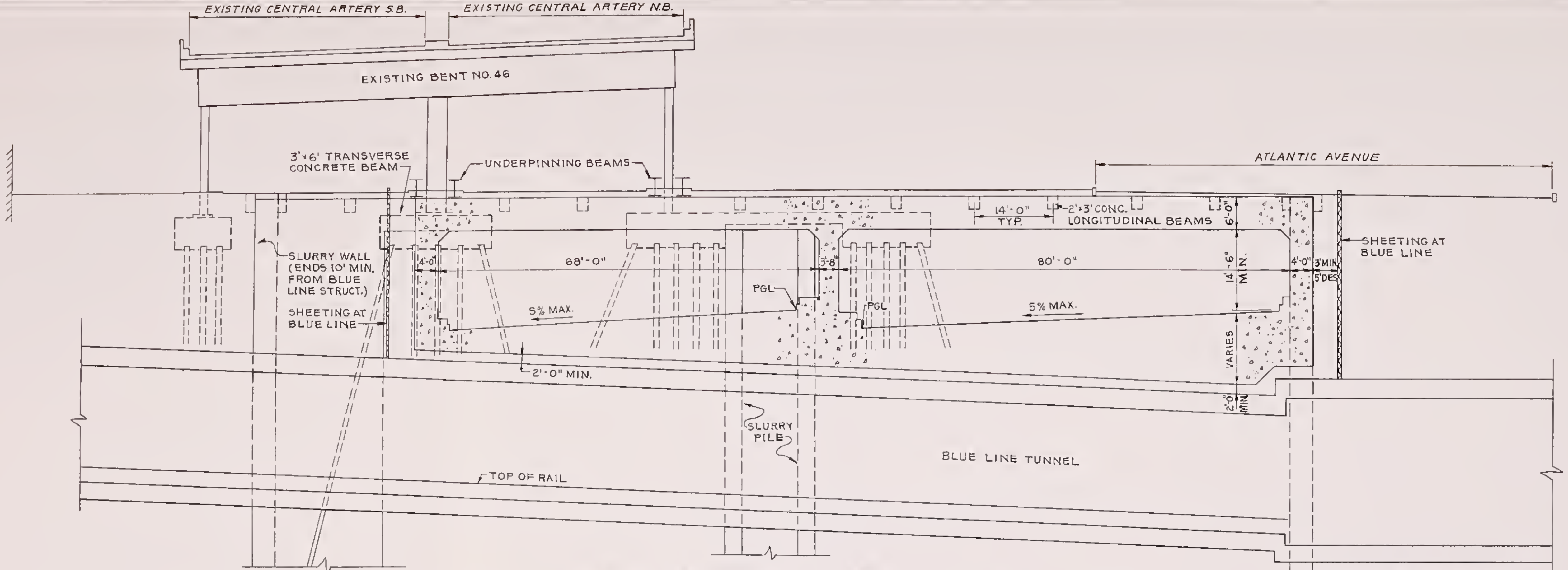
Sta. 160 + 00 to Portal

Figure 26

Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections Central Area

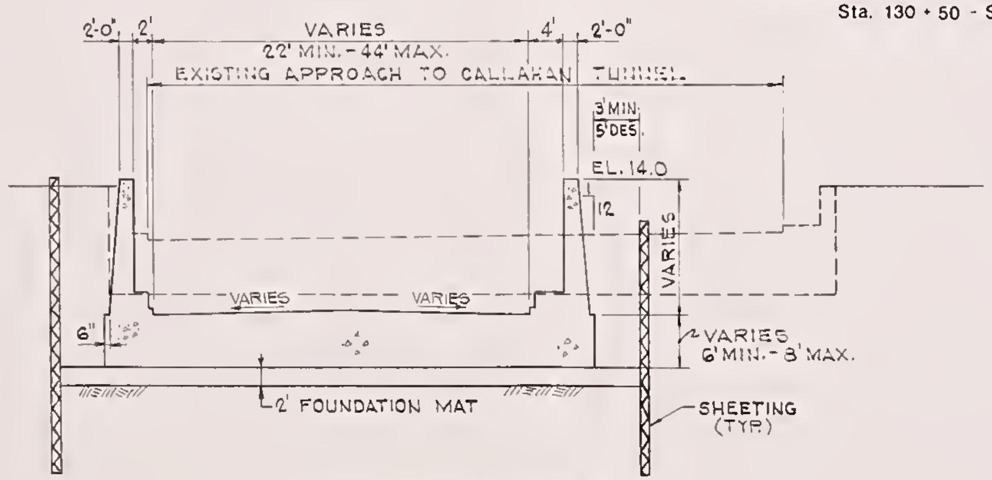
EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet

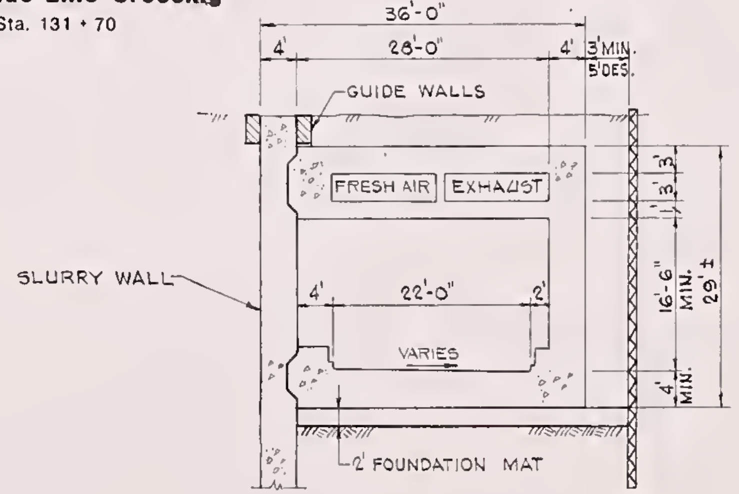


Central Artery - Blue Line Crossing

Sta. 130 + 50 - Sta. 131 + 70



Callahan Tunnel Approach



Ramp SA-CS

NOTE:

TYPICAL SECTIONS FOR RAMPS CS-P, ST-CN, CS-CT, (1 LANE TUNNEL TOP & BOTTOM VENTING, AND 1 LANE BOAT SECTION) ARE SHOWN ON FIGURE 19.

Figure 27

Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet

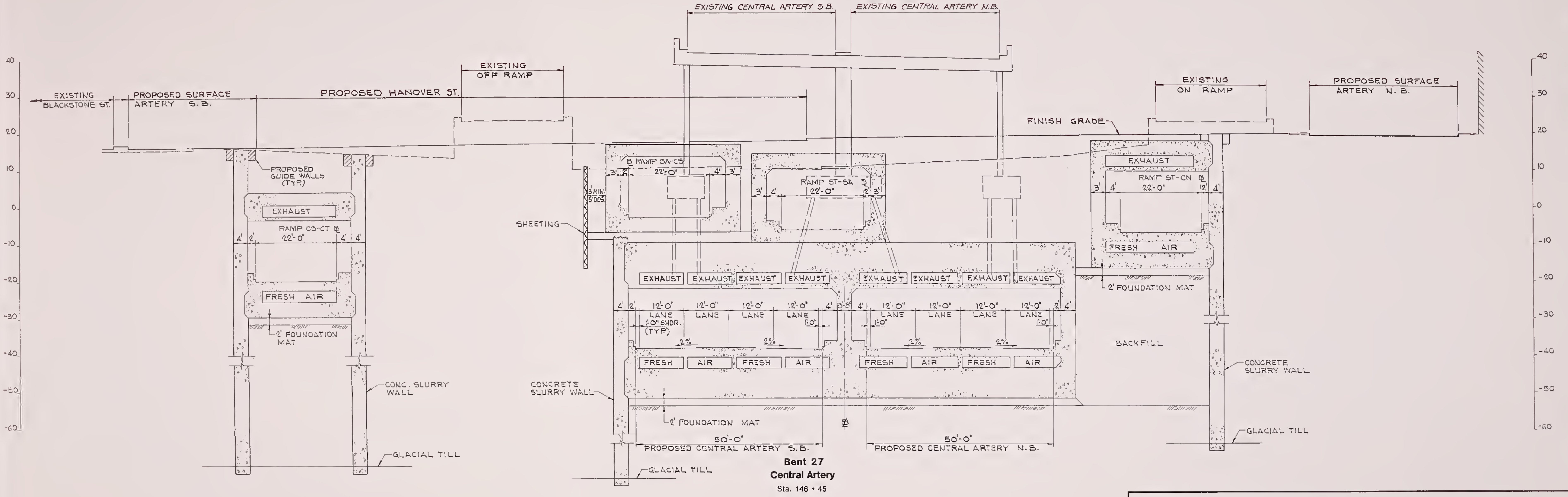
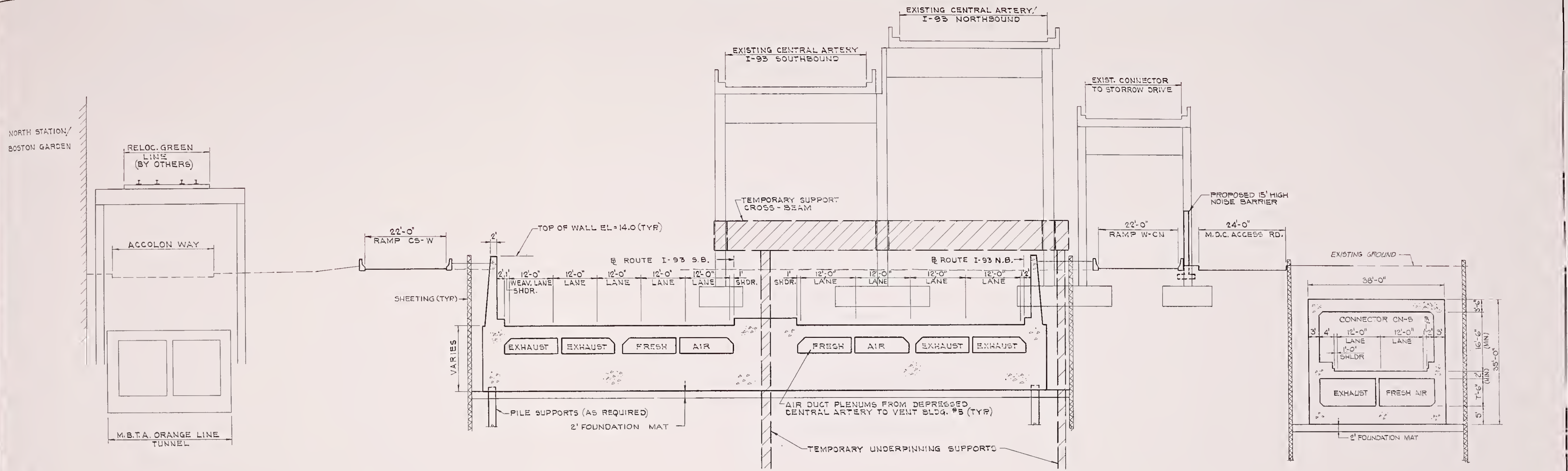


Figure 28

Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

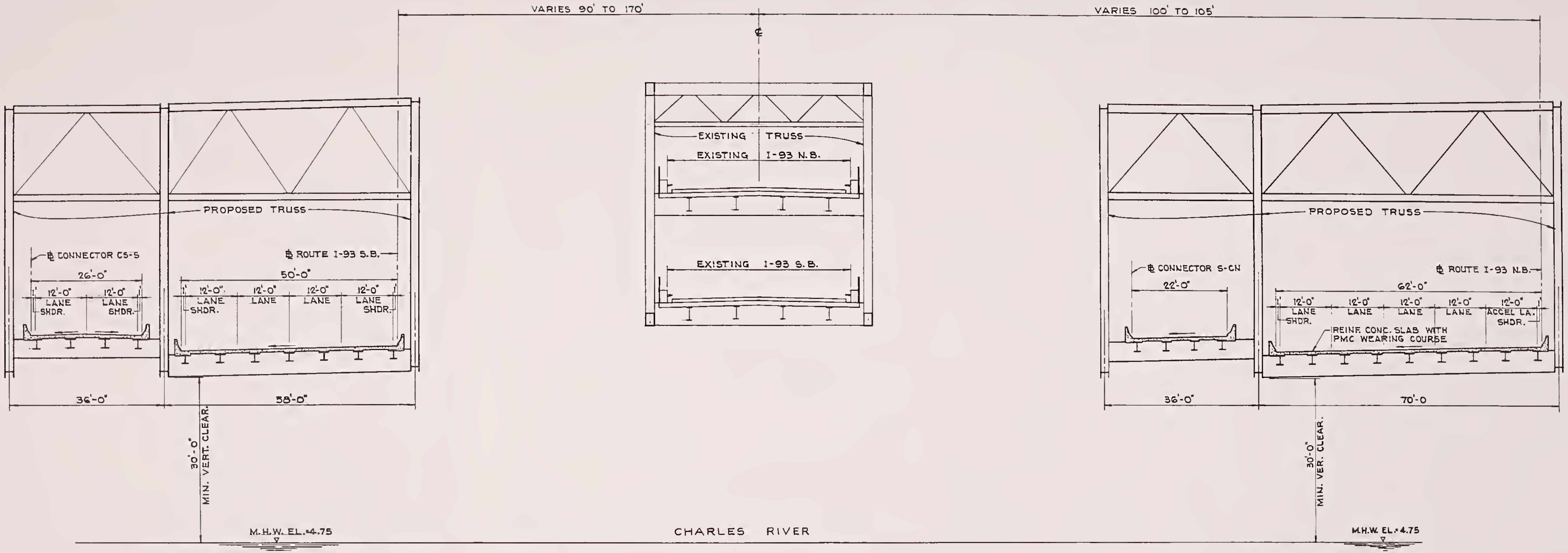
0 5 10 20 Feet



Central Artery/I-93
Sta. 164 + 80 - Sta. 167 + 00

Figure 29
Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



Route I-93 Southbound
Sta. 172 + 75 - Sta. 176 + 75

Route I-93 Northbound
Sta. 171 + 85 - Sta. 175 + 60

Figure 29A
Preferred Alternative (5A Modified 4 Lane Tunnel) – Typical Sections North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet

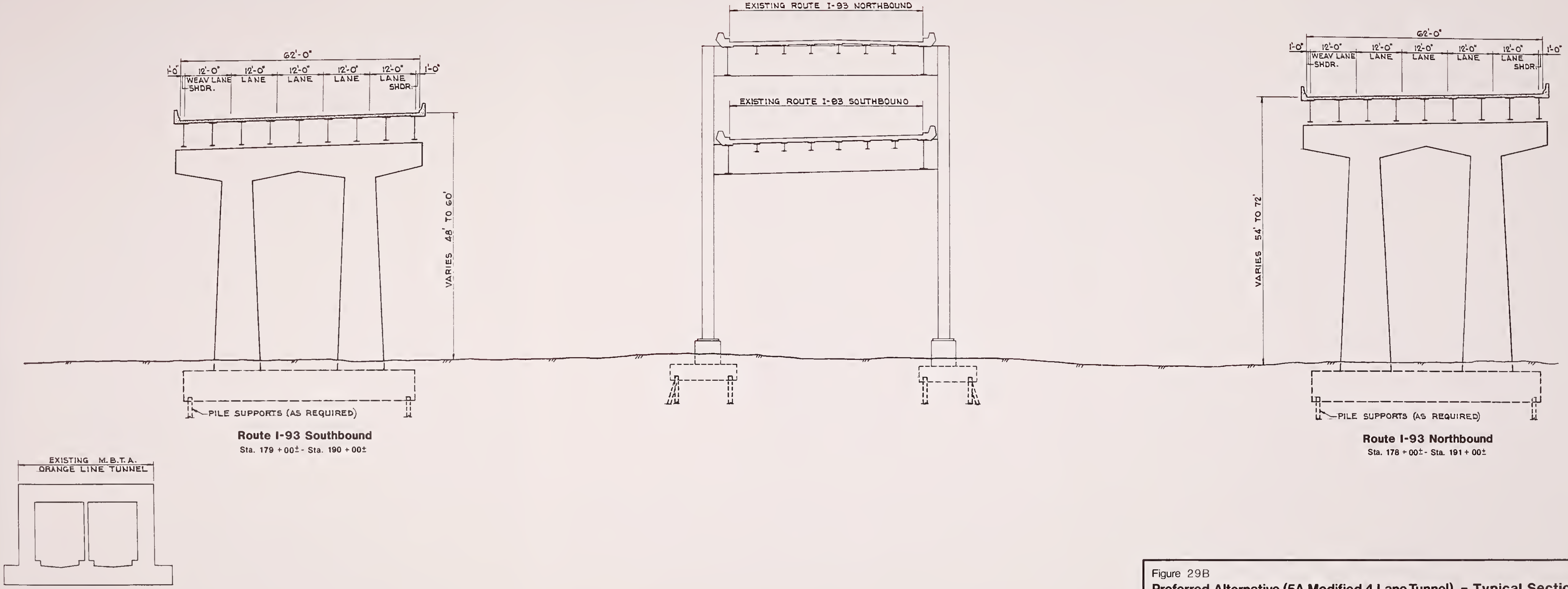
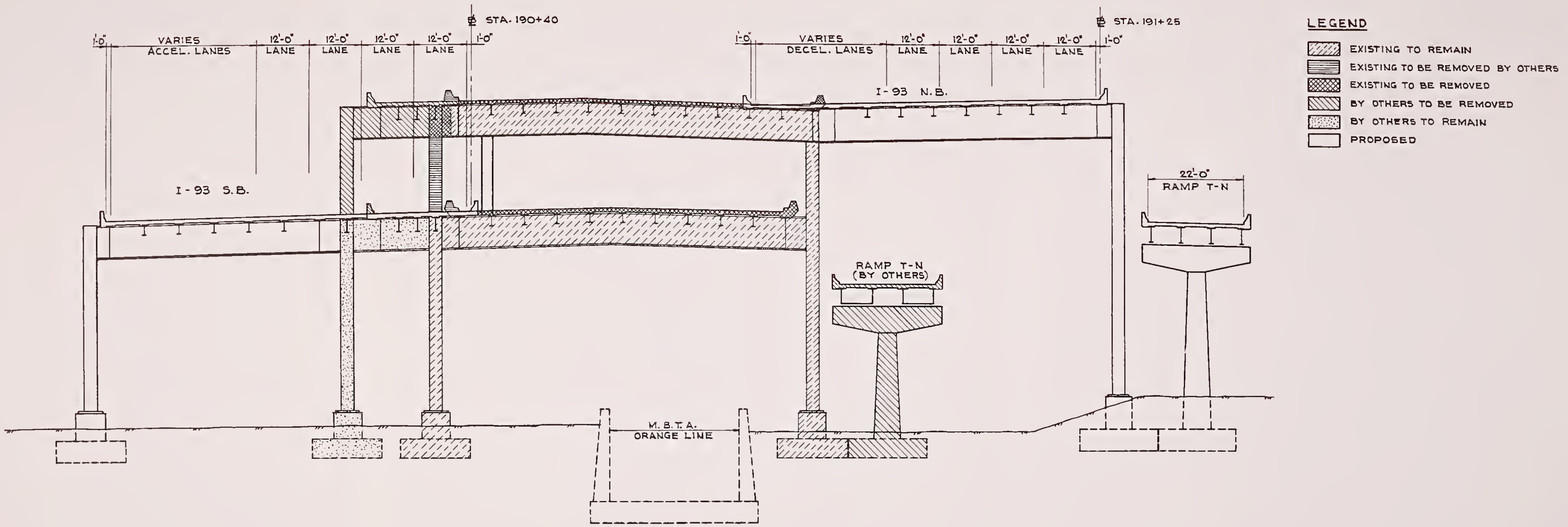


Figure 29B
Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet



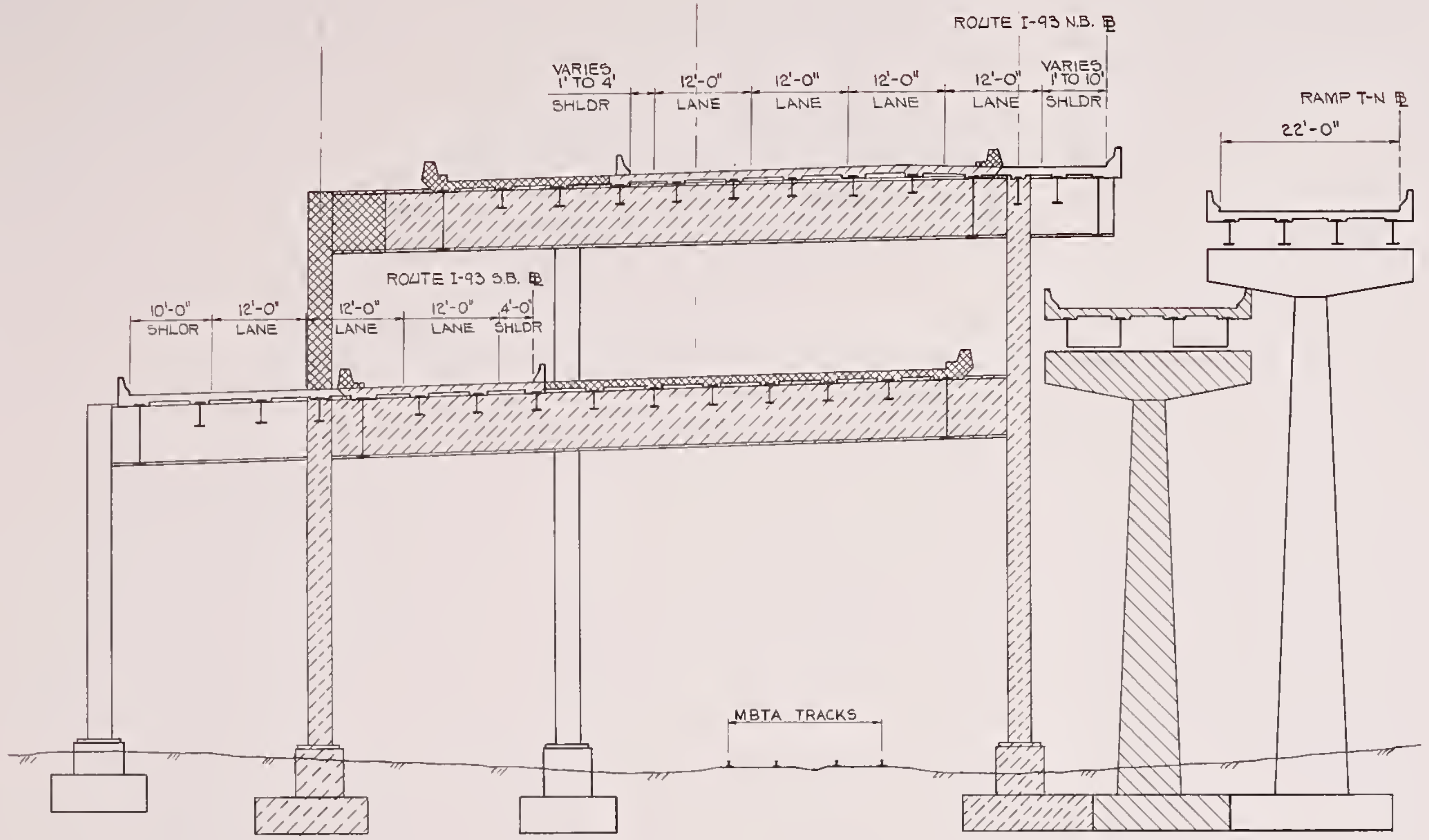
Bent 15 B

Figure 29C

Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

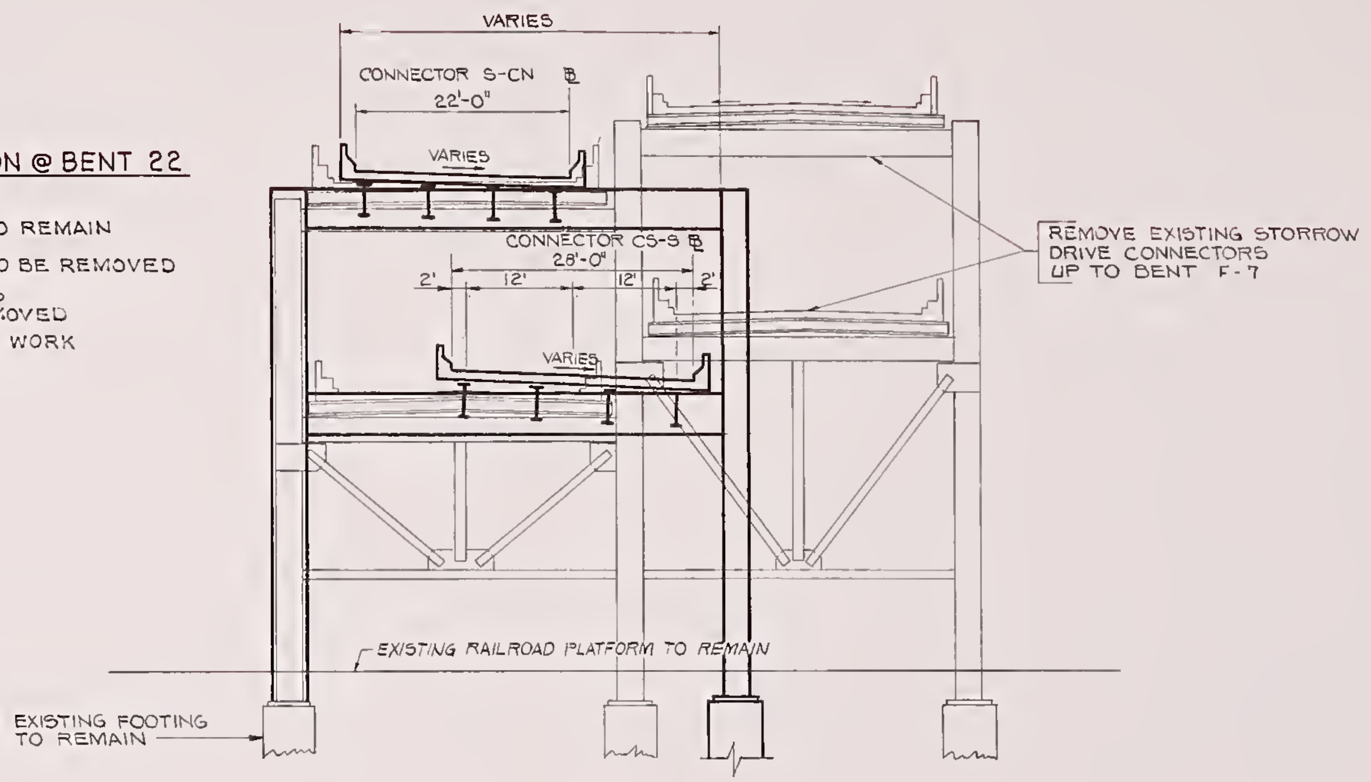
0 5 10 20 Feet



LEGEND - SECTION @ BENT 22

- EXISTING TO REMAIN
- EXISTING TO BE REMOVED
- BY OTHERS TO BE REMOVED
- PROPOSED WORK

Bent 22 - I-93
Sta. 197 + 00± I-93 Northbound
Sta. 196 + 00± I-93 Southbound

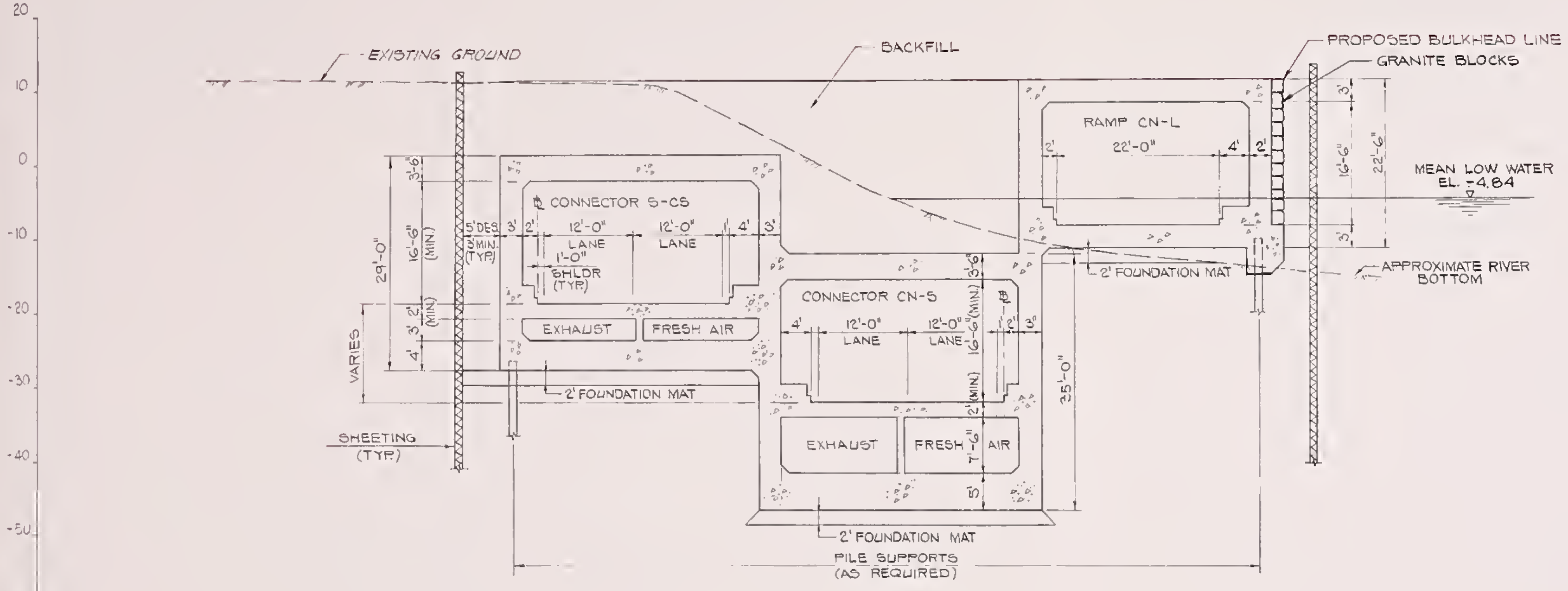


Storrow Drive Ramps
Sta. 7 + 50 - Sta. 10 + 00

Figure 30
Preferred Alternative (5A Modified 4 Lane Tunnel) – Typical Sections North Area

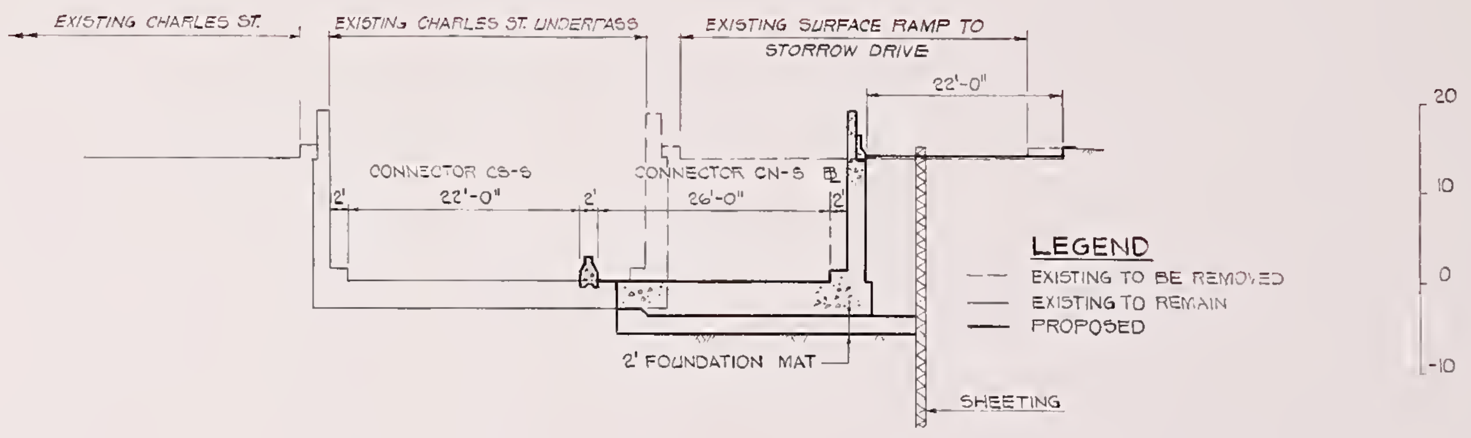
EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet



Connectors S-CS, CN-S & CN-L

Sta. 80+00 - Sta. 84+00



Connector CN-S

Sta. 94 + 00 - Sta. 97 + 00

Figure 31

Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

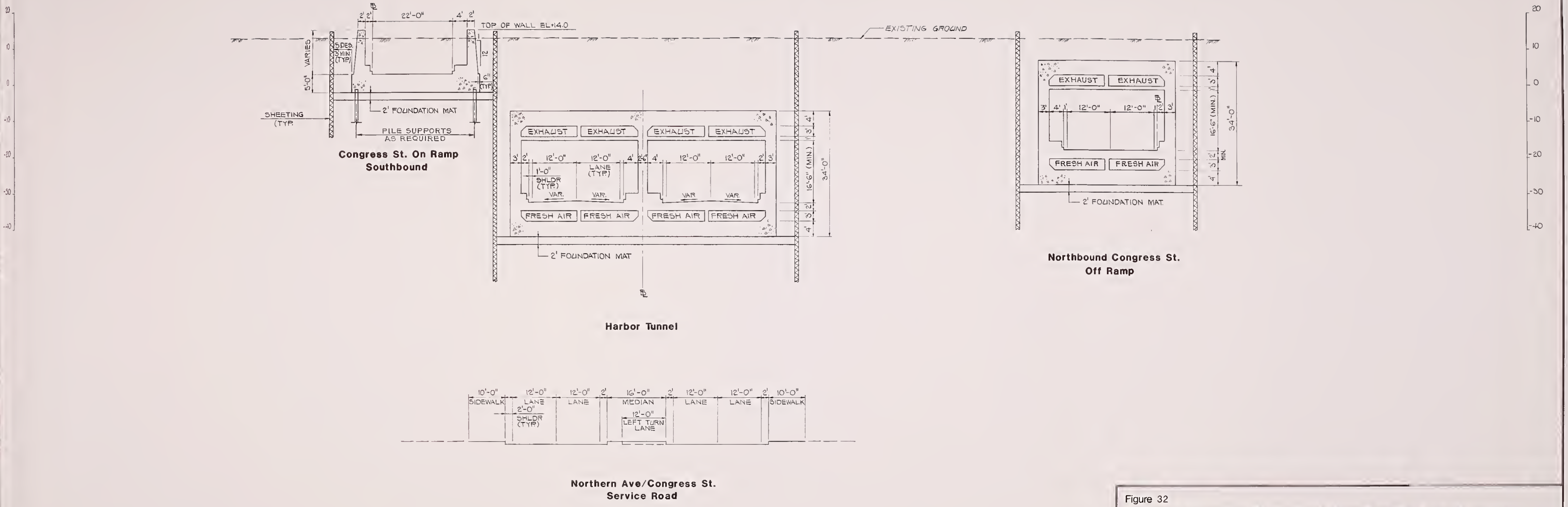
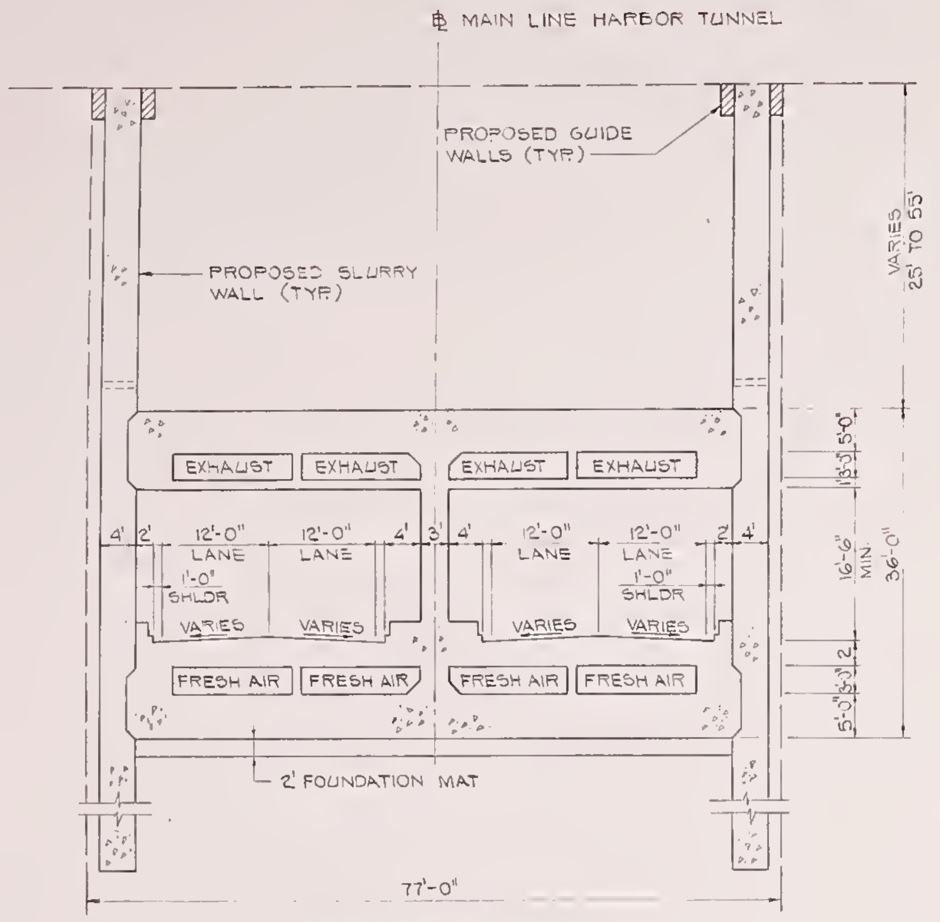


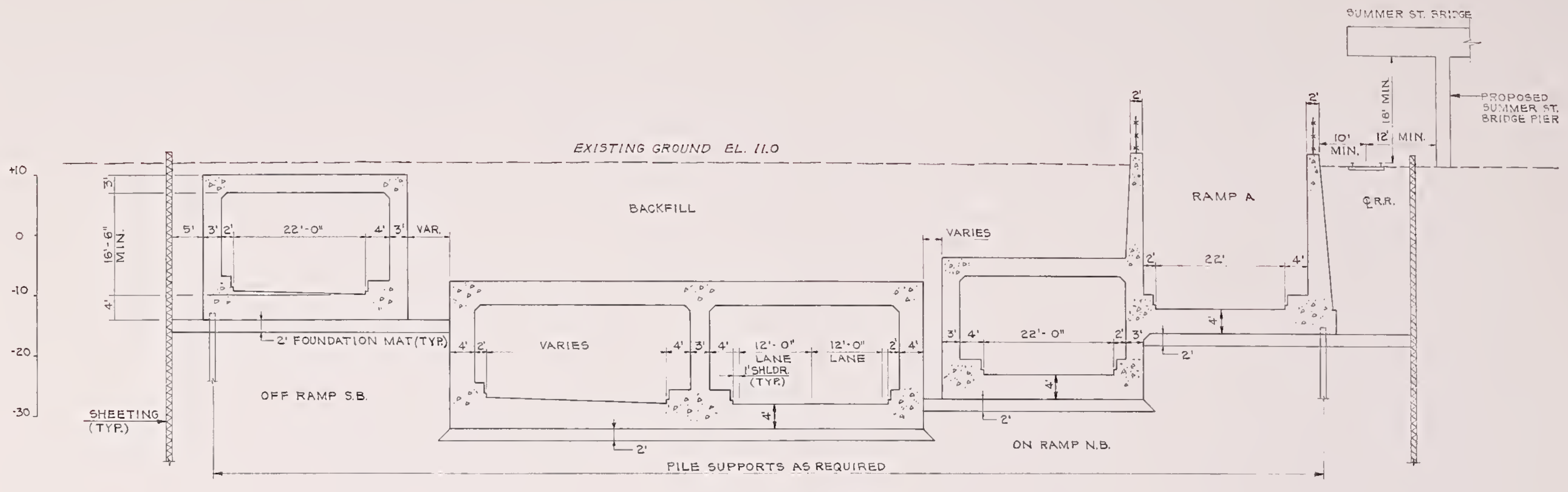
Figure 32

Preferred Alternative (5A Modified 4 Lane Tunnel) – Typical Sections South Boston Area

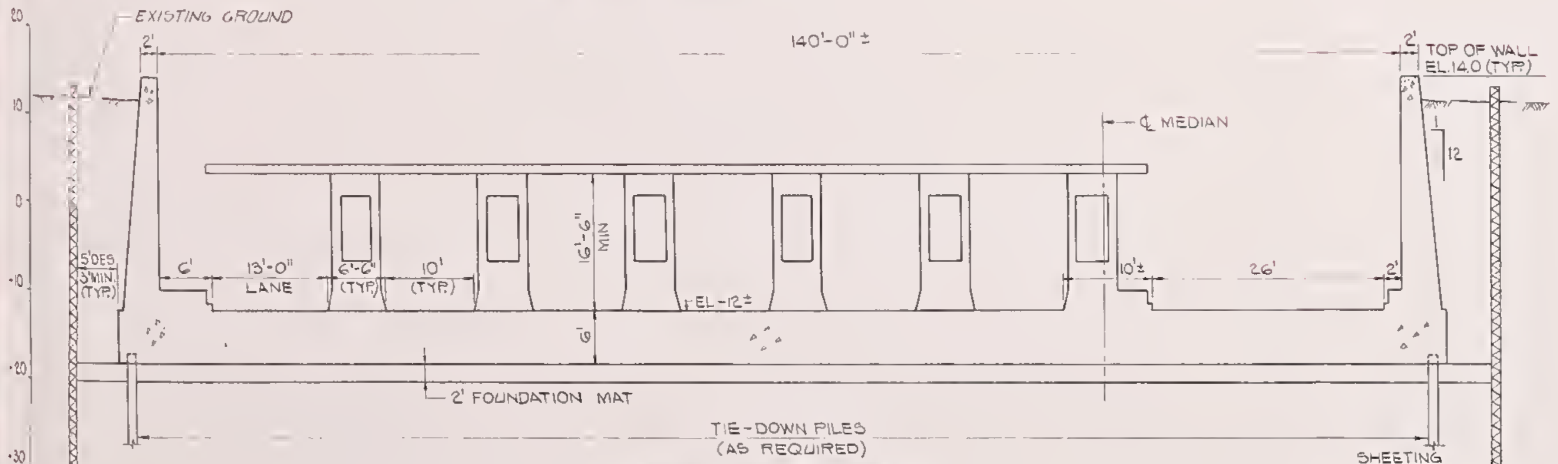
EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



Mainline Harbor Tunnel
Slurry Wall Construction
Sta. 82+00 - Sta. 92+00



Harbor Tunnel
Sta. 62+00±

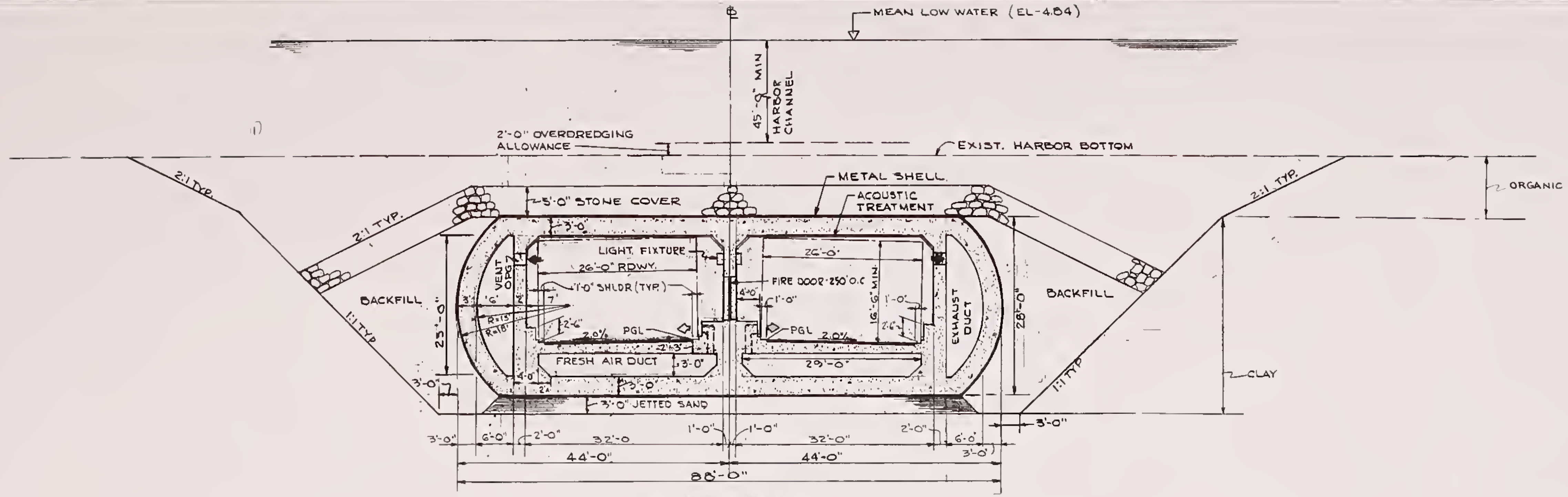


Mainline Airport Tunnel
Toll Plaza - South Boston

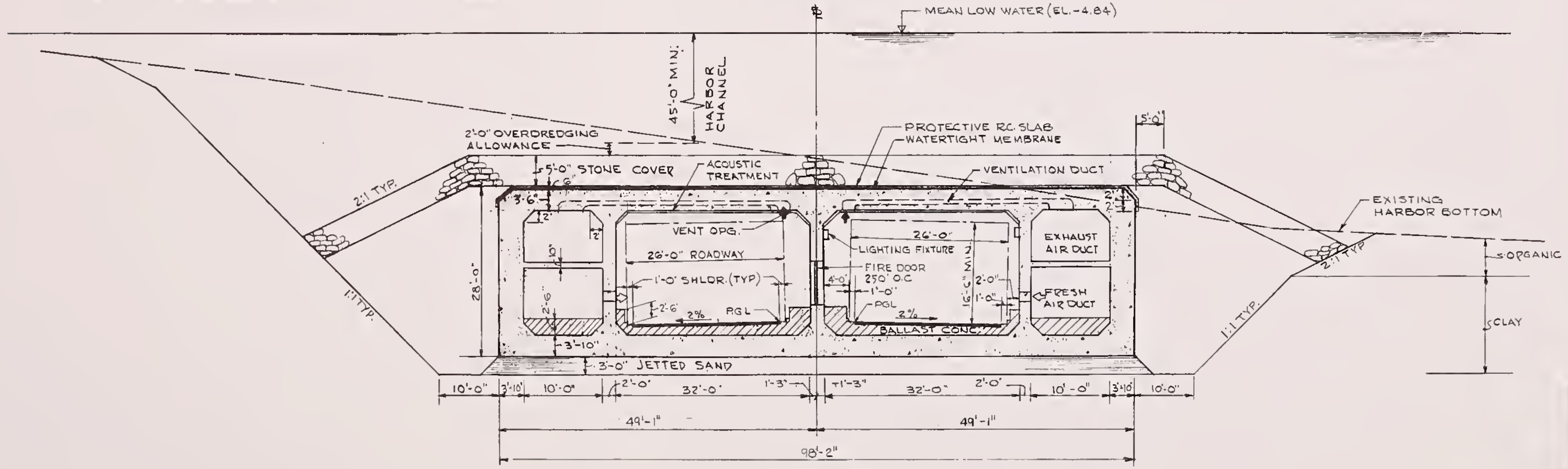
Figure 33

Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections South Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



Harbor Tunnel
Metal Sunken Tube

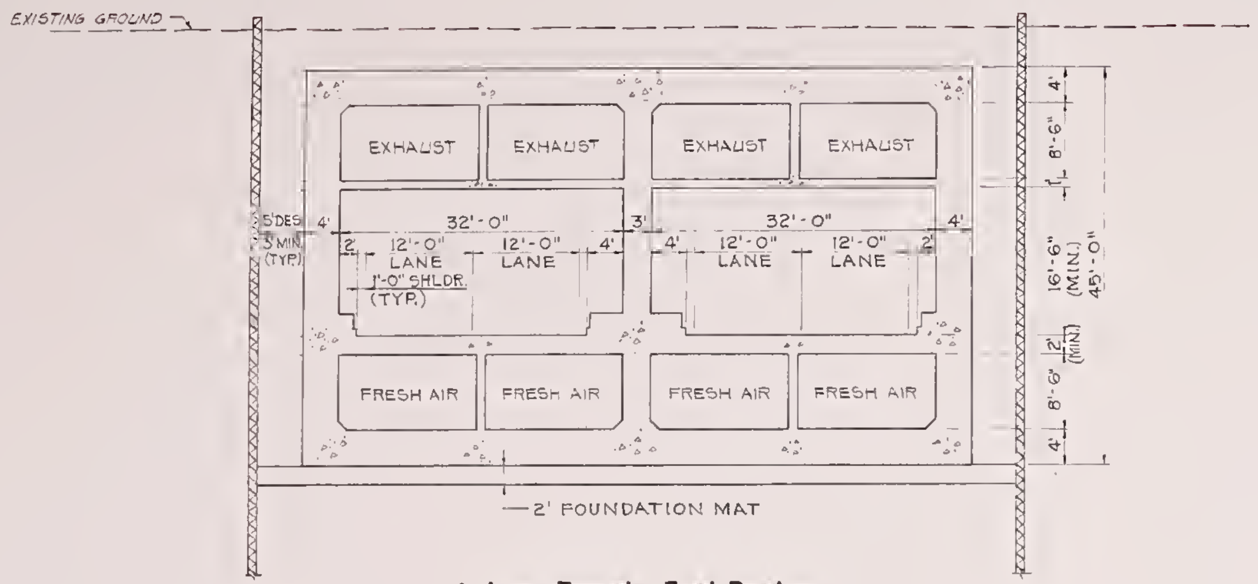


Harbor Tunnel
Concrete Sunken Tube

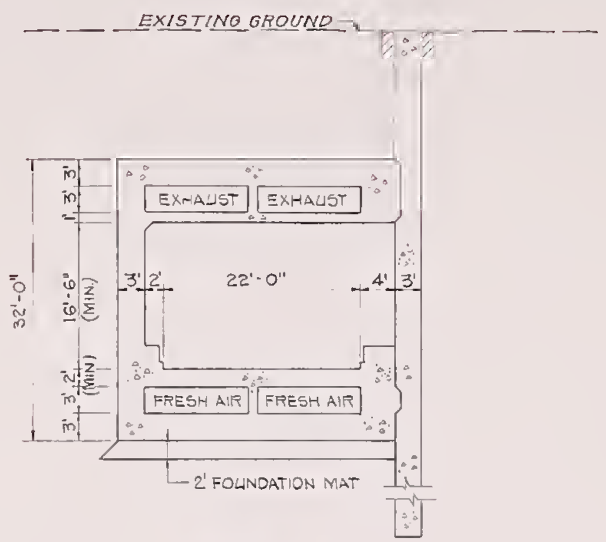
Figure 34
Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections Boston Harbor Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

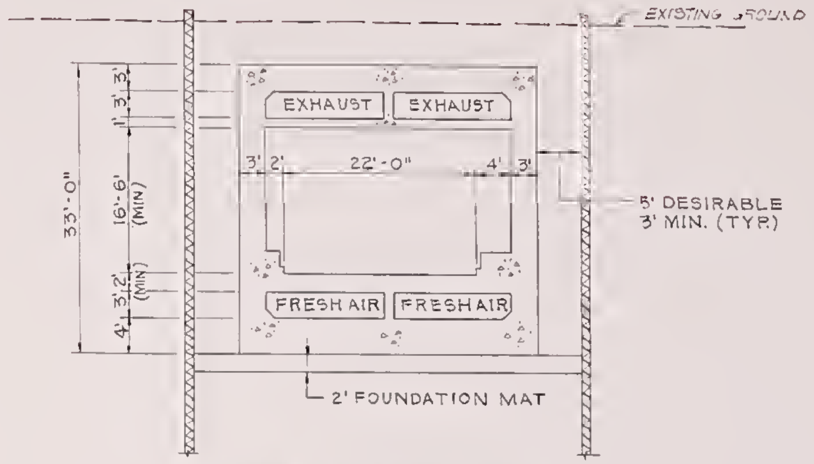
0 5 10 20 Feet



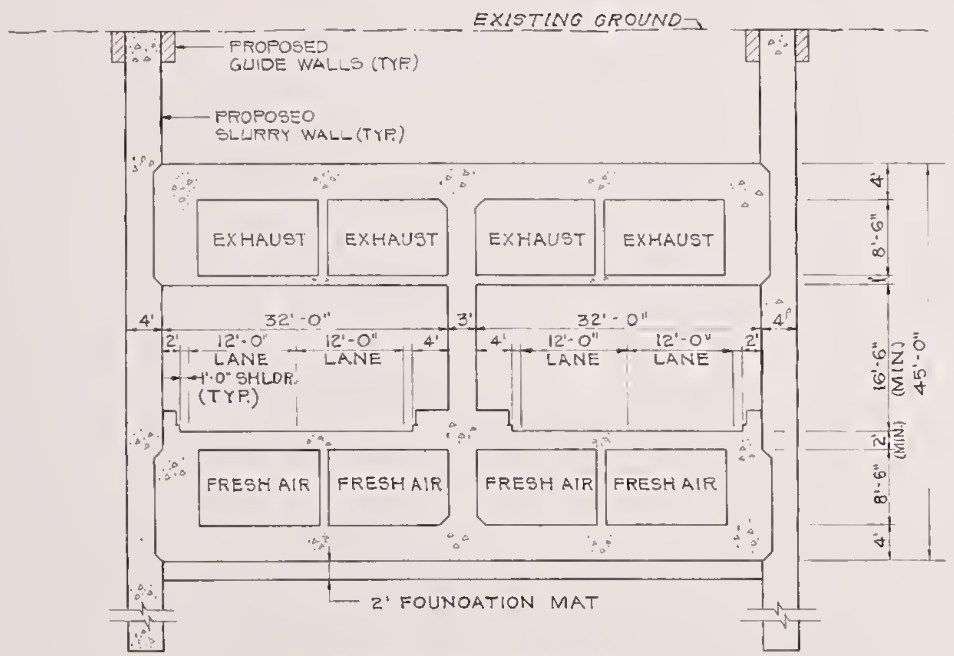
4 Lane Tunnel - East Boston



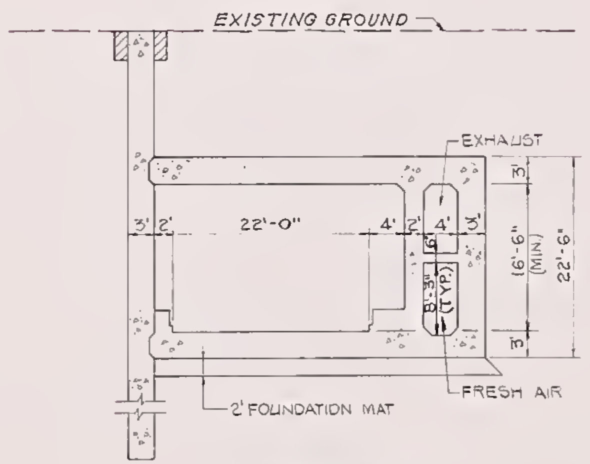
Ramp AP-2
Sta. 8+50± - Sta. 11+50±



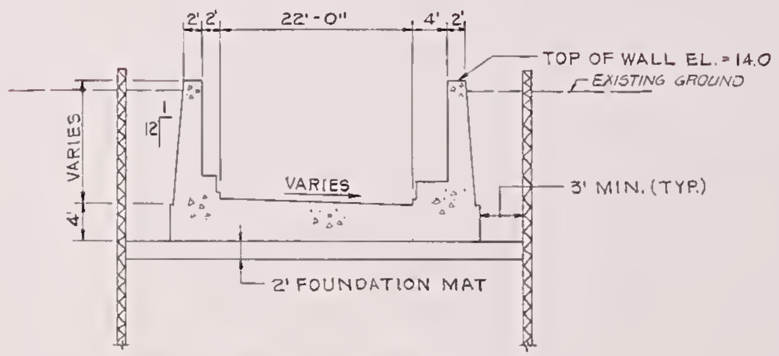
Ramps AP-1, AP-2 & AP-4



4 Lane Tunnel - East Boston



Ramp AP-3
Sta. 9+00± - Sta. 14+50±

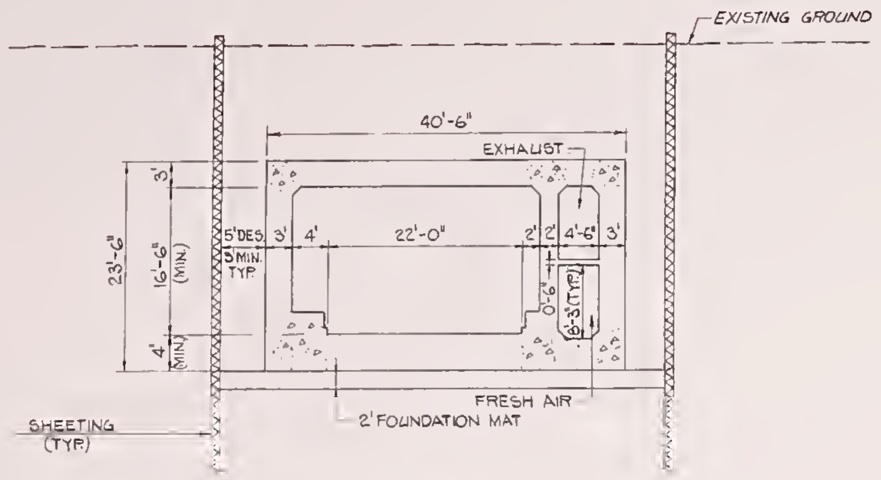


Ramps AP-1, AP-2, AP-4 & 1A-1

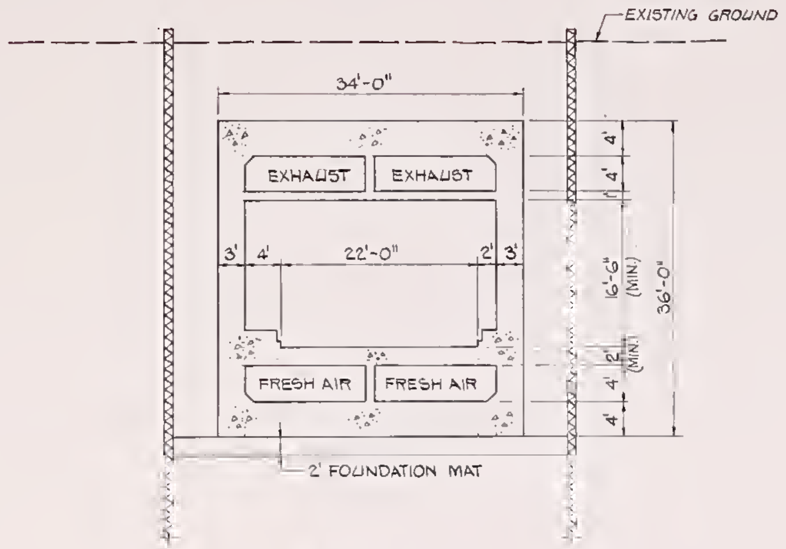
Figure 35
Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet

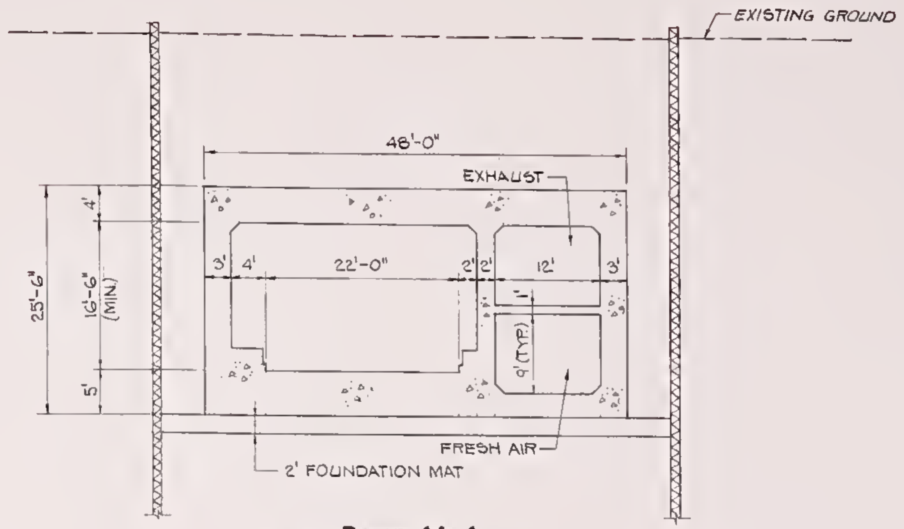


Ramp AP-3
Sta. 14+50± To Portal

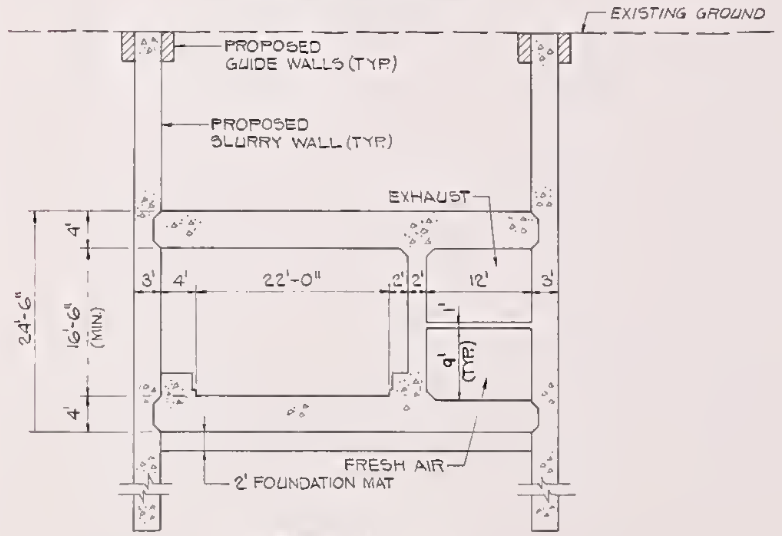


Ramp 1A-1
Sta. 200 + 00± - Sta. 211 + 00±

NOTE: AIR DUCTS TRANSITION FROM TOP & BOTTOM TO SIDE BETWEEN STA. 211 + OO & 213 + OO



Ramp 1A-1
Sta. 213+00± - Sta. 215+00±.
Sta. 217+20± - Sta. 223+00±

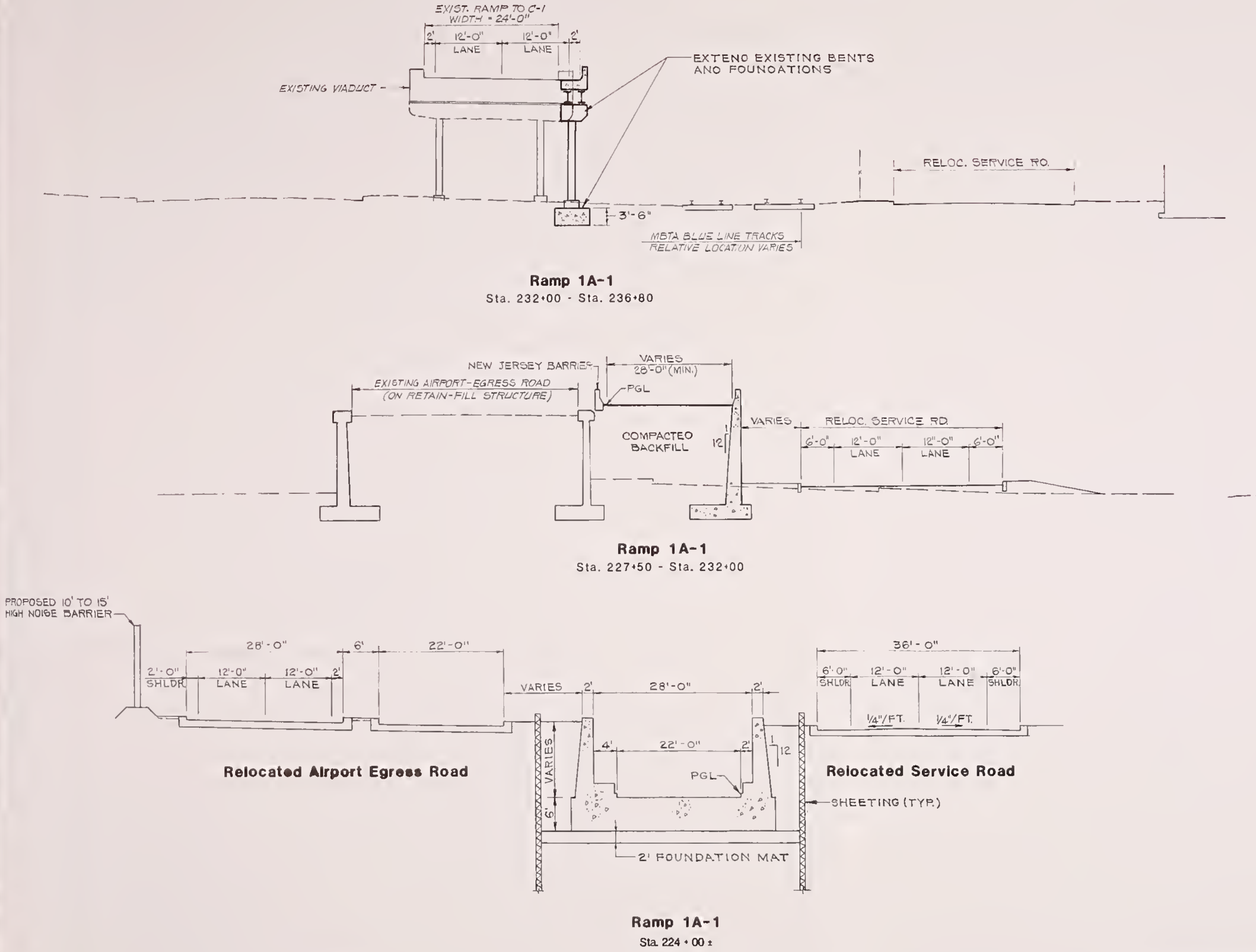


Ramp 1A-1
Sta. 215+00± - Sta. 217+20±

Figure 36
Preferred Alternative (5A Modified 4 Lane Tunnel) – Typical Sections East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet



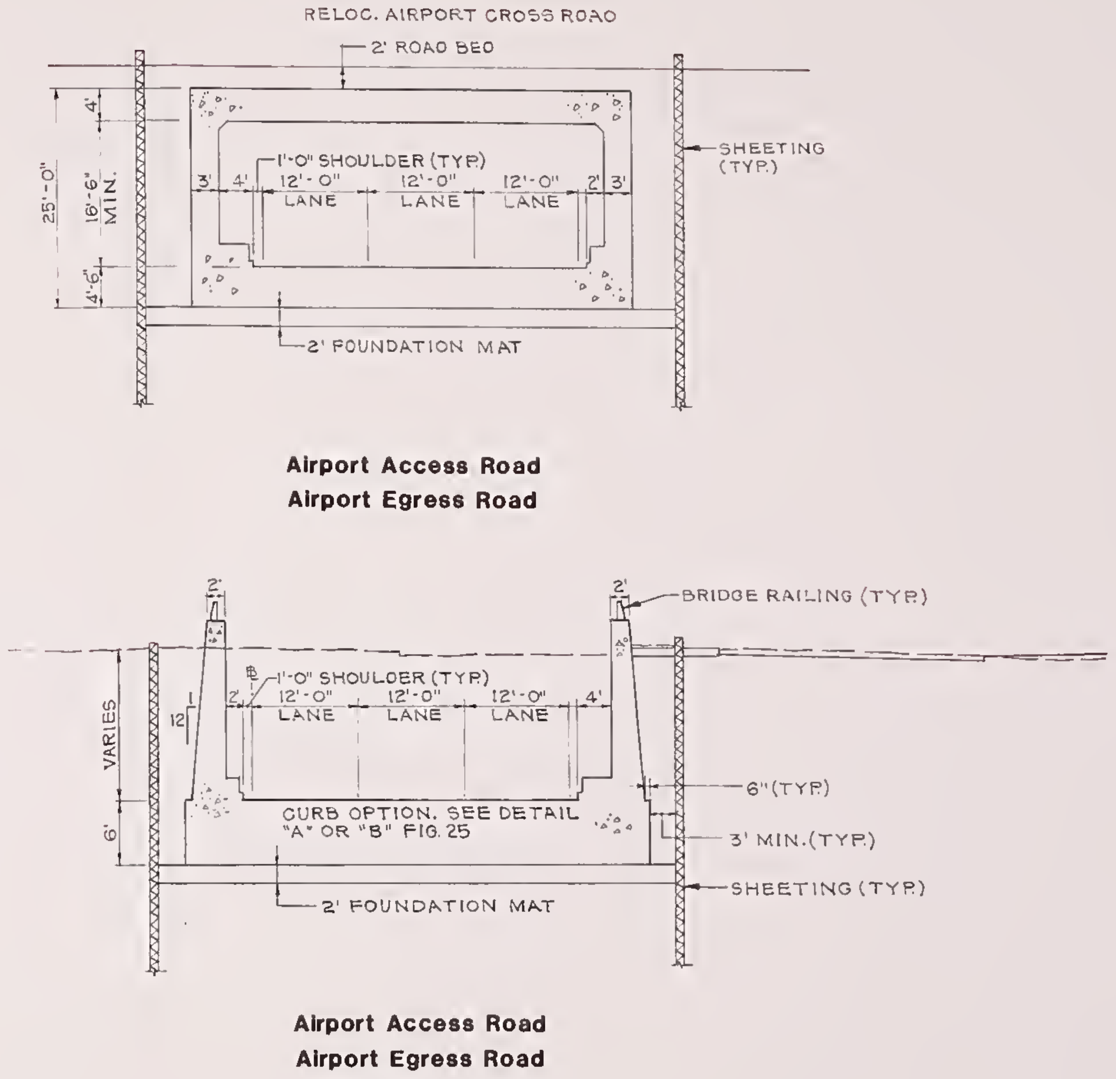


Figure 37
Preferred Alternative (5A Modified 4 Lane Tunnel) - Typical Sections East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

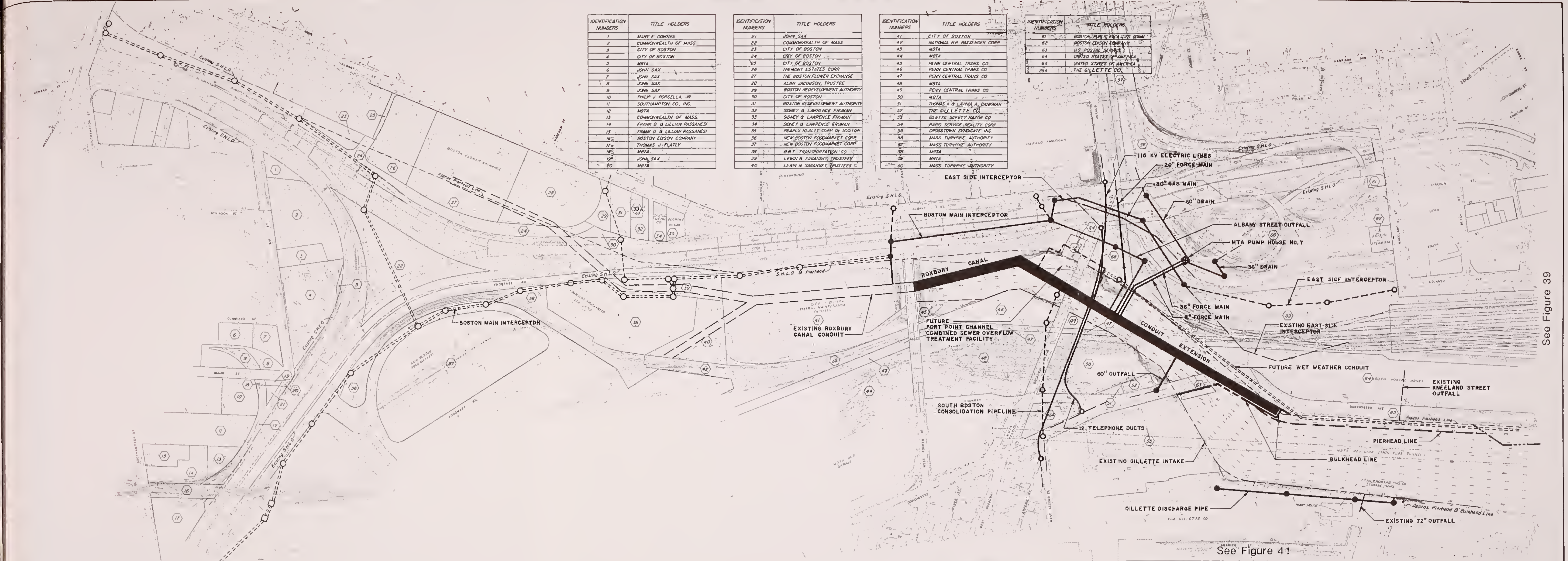
0 5 10 20 Feet

IDENTIFICATION NUMBERS	TITLE HOLDERS
1	MARY E DOWNES
2	COMMONWEALTH OF MASS
3	CITY OF BOSTON
4	CITY OF BOSTON
5	MBTA
6	JOHN SAX
7	JOHN SAX
8	JOHN SAX
9	JOHN SAX
10	PHILIP J PORCELLA, JR
11	SOUTHAMPTON CO, INC.
12	MBTA
13	COMMONWEALTH OF MASS
14	FRANK D & LILLIAN PASSANESI
15	FRANK D & LILLIAN PASSANESI
16	BOSTON EDISON COMPANY
17	THOMAS J FLATLY
18	MBTA
19	JOHN SAX
20	MBTA

IDENTIFICATION NUMBERS	TITLE HOLDERS
21	JOHN SAX
22	COMMONWEALTH OF MASS
23	CITY OF BOSTON
24	CITY OF BOSTON
25	CITY OF BOSTON
26	TREMONT ESTATES CORP
27	THE BOSTON FLOWER EXCHANGE
28	ALAN JACOBSON, TRUSTEE
29	BOSTON REDEVELOPMENT AUTHORITY
30	CITY OF BOSTON
31	BOSTON REDEVELOPMENT AUTHORITY
32	SIDNEY & LAWRENCE FRUMAN
33	SIDNEY & LAWRENCE FRUMAN
34	SIDNEY & LAWRENCE FRUMAN
35	PEARLS REALTY CORP OF BOSTON
36	NEW BOSTON FOODMARKET CORP
37	NEW BOSTON FOODMARKET CORP
38	B & T TRANSPORTATION CO
39	LEWIN & SAGANSKY, TRUSTEES
40	LEWIN & SAGANSKY, TRUSTEES

IDENTIFICATION NUMBERS	TITLE HOLDERS
41	CITY OF BOSTON
42	NATIONAL RR PASSENGER CORP
43	MBTA
44	MBTA
45	PENN CENTRAL TRANS CO
46	PENN CENTRAL TRANS CO
47	PENN CENTRAL TRANS CO
48	MBTA
49	PENN CENTRAL TRANS CO
50	MBTA
51	THOMAS A & LAVINIA A BANKMAN
52	THE GILLETTE CO.
53	GILETTE SAFETY RAZOR CO
54	RAPID SERVICE REALITY CORP
55	CROSSTOWN SYNDICATE INC
56	MASS TURNPIKE AUTHORITY
57	MASS TURNPIKE AUTHORITY
58	MBTA
59	MBTA
60	MASS TURNPIKE AUTHORITY

IDENTIFICATION NUMBERS	TITLE HOLDERS
61	BOSTON PUBLIC FACILITIES COMM
62	BOSTON EDISON COMPANY
63	U.S. POSTAL SERVICE
64	UNITED STATES OF AMERICA
65	UNITED STATES OF AMERICA
264	THE GILLETTE CO.



See Figure 39

See Figure 41

LEGEND

- PROPOSED UTILITY
- PROPOSED UTILITY BY OTHERS
- EXISTING UTILITY
- PROPOSED STRUCTURE
- PROPOSED STRUCTURE BY OTHERS

Figure 38

Preferred Alternative (5A Modified 4 Lane Tunnel) Property and Utilities Plan - South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 100 200 400 Feet

IDENTIFICATION NUMBERS	TITLE HOLDERS
39 S	ROBERT C LINNELL
40 S	ROBERT C LINNELL
41 S	M B T A
42 S	M B T A
43 S	JOHN E BENNETT
44 S	CITY OF BOSTON
45 S	BOSTON REDEVELOPMENT AUTHORITY
46 S	EDWARD M SWARTZ
47 S	LAWRENCE A CREAVES
48 S	PHILIP Y DENORMANDIE
49 S	LINZEE J COOLIDGE
50 S	LUCY F MANOZI
51 S	PAUL A KRAMER
52 S	PAUL A KRAMER
53 S	BESSIE RESNICK
54 S	BOSTONIAN HOTEL ASSOC
55 S	LINZEE J COOLIDGE
56 S	ARTHUR M WINN
57 S	BOSTON REDEVELOPMENT AUTHORITY
58 S	BOSTON REDEVELOPMENT AUTHORITY

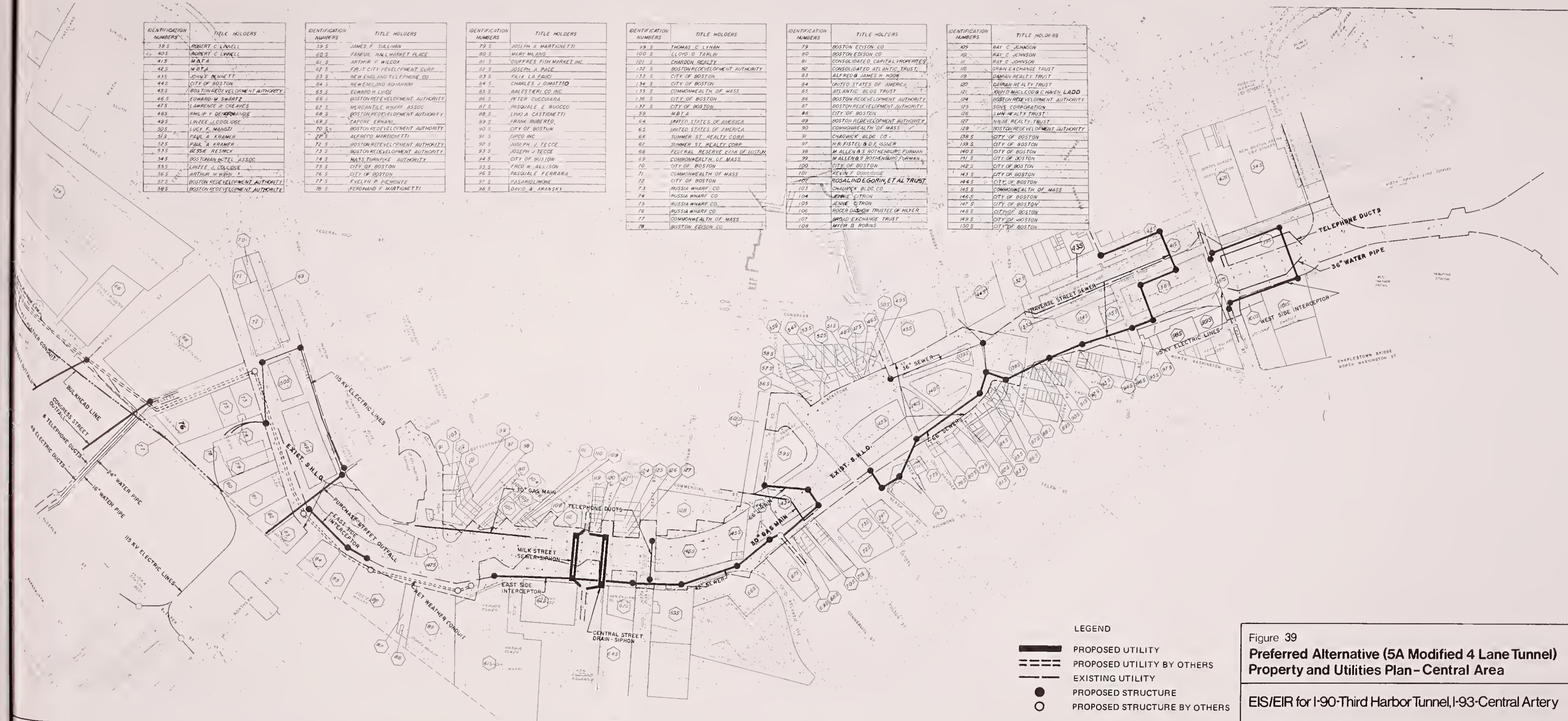
IDENTIFICATION NUMBERS	TITLE HOLDERS
59 S	JAMES F SULLIVAN
60 S	FANEUIL HALL MARKET PLACE
61 S	ARTHUR P WILCOX
62 S	FIRST CITY DEVELOPMENT CORP
63 S	NEW ENGLAND TELEPHONE CO
64 S	NEW ENGLAND AQUARIUM
65 S	EDWARD H LINDE
66 S	BOSTON REDEVELOPMENT AUTHORITY
67 S	MERCANTILE WHARF ASSOC
68 S	BOSTON REDEVELOPMENT AUTHORITY
69 S	CAPONE ERNANI
70 S	BOSTON REDEVELOPMENT AUTHORITY
71 S	ALFREDO MARTIGNETTI
72 S	BOSTON REDEVELOPMENT AUTHORITY
73 S	BOSTON REDEVELOPMENT AUTHORITY
74 S	MASS TURNPIKE AUTHORITY
75 S	CITY OF BOSTON
76 S	CITY OF BOSTON
77 S	EVELYN P PIEMONTE
78 S	FERDINAND F MARTIGNETTI

IDENTIFICATION NUMBERS	TITLE HOLDERS
79 S	JOSEPH A MARTIGNETTI
80 S	MARY MILANO
81 S	CIUFFRES FISH MARKET INC
82 S	JOSEPH A PACE
83 S	FILIX LA FAUCI
84 S	CHARLES J DIMATTEO
85 S	BALESTIERI CO INC
86 S	PETER CUCCUIARA
87 S	PASQUALE E RUOCCO
88 S	LINO A CASTIGNETTI
89 S	FRANK RUBERTO
90 S	CITY OF BOSTON
91 S	CIPCO INC
92 S	JOSEPH J TECCE
93 S	JOSEPH J TECCE
94 S	CITY OF BOSTON
95 S	FRED W ALLISON
96 S	PASQUALE FERRARA
97 S	PASARIO LIMONE
98 S	DAVID A ARANSKY

IDENTIFICATION NUMBERS	TITLE HOLDERS
99 S	THOMAS C LYNAH
100 S	LLOYD O TARLIN
101 S	CHARDON REALTY
132 S	BOSTON REDEVELOPMENT AUTHORITY
133 S	CITY OF BOSTON
134 S	CITY OF BOSTON
135 S	COMMONWEALTH OF MASS
136 S	CITY OF BOSTON
137 S	CITY OF BOSTON
59	M B T A
64	UNITED STATES OF AMERICA
65	UNITED STATES OF AMERICA
66	SUMMER ST REALTY CORP
67	SUMMER ST REALTY CORP
68	FEDERAL RESERVE BANK OF BOSTON
69	COMMONWEALTH OF MASS
70	CITY OF BOSTON
71	COMMONWEALTH OF MASS
72	CITY OF BOSTON
73	RUSSIA WHARF CO
74	RUSSIA WHARF CO
75	RUSSIA WHARF CO
76	RUSSIA WHARF CO
77	COMMONWEALTH OF MASS
78	BOSTON EDISON CO

IDENTIFICATION NUMBERS	TITLE HOLDERS
79	BOSTON EDISON CO
80	BOSTON EDISON CO
81	CONSOLIDATED CAPITAL PROPERTIES
82	CONSOLIDATED ATLANTIC TRUST
83	ALFRED & JAMES H HOOK
84	UNITED STATES OF AMERICA
85	ATLANTIC BLDG TRUST
86	BOSTON REDEVELOPMENT AUTHORITY
87	BOSTON REDEVELOPMENT AUTHORITY
88	CITY OF BOSTON
89	BOSTON REDEVELOPMENT AUTHORITY
90	COMMONWEALTH OF MASS
91	CHADWICK BLDG CO
97	H R FISTEL & D F ISSNER
98	M ALLEN & S ROTHENBURG FURMAN
99	M ALLEN & S ROTHENBURG FURMAN
100	CITY OF BOSTON
101	KEVIN F DONOGHUE
102	ROSALIND EGORIN, ET AL TRUST
103	CHADWICK BLDG CO
104	JENNIE CITRON
105	JENNIE CITRON
106	ROGER DASHOW TRUSTEE OF HILYER
107	BROAD EXCHANGE TRUST
108	MYER D ROBINS

IDENTIFICATION NUMBERS	TITLE HOLDERS
109	RAY C JOHNSON
110	RAY C JOHNSON
111	RAY C JOHNSON
112	GRAIN EXCHANGE TRUST
119	DAMIAN REALTY TRUST
120	DAMIAN REALTY TRUST
121	JOHN D MACLEOD & C HAVEN LADD
124	BOSTON REDEVELOPMENT AUTHORITY
125	SONS CORPORATION
126	SMN REALTY TRUST
127	NINJA REALTY TRUST
128	BOSTON REDEVELOPMENT AUTHORITY
138 S	CITY OF BOSTON
139 S	CITY OF BOSTON
140 S	CITY OF BOSTON
141 S	CITY OF BOSTON
142 S	CITY OF BOSTON
143 S	CITY OF BOSTON
144 S	CITY OF BOSTON
145 S	COMMONWEALTH OF MASS
146 S	CITY OF BOSTON
147 S	CITY OF BOSTON
148 S	CITY OF BOSTON
149 S	CITY OF BOSTON
150 S	CITY OF BOSTON



See Figure 40

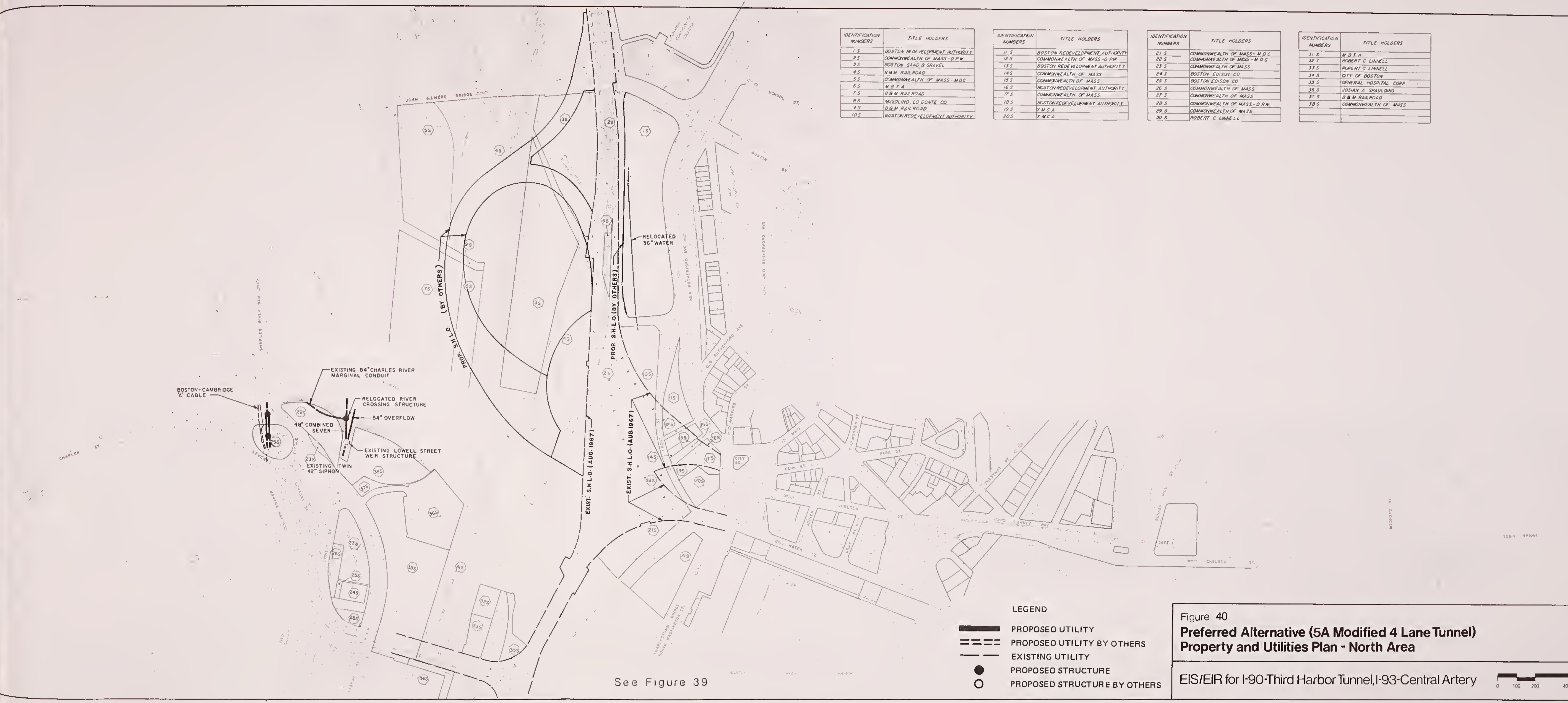
LEGEND

- PROPOSED UTILITY
- PROPOSED UTILITY BY OTHERS
- EXISTING UTILITY
- PROPOSED STRUCTURE
- PROPOSED STRUCTURE BY OTHERS

Figure 39
Preferred Alternative (5A Modified 4 Lane Tunnel)
Property and Utilities Plan - Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 100 200 400 Feet



IDENTIFICATION NUMBERS	TITLE HOLDERS
1 S	BOSTON REDEVELOPMENT AUTHORITY
2 S	COMMONWEALTH OF MASS - D P W
3 S	BOSTON SAND & GRAVEL
4 S	B & M RAILROAD
5 S	COMMONWEALTH OF MASS - MDC
6 S	M B T A
7 S	B & M RAILROAD
8 S	MUSOLINO LO CONTE CO.
9 S	B & M RAILROAD
10 S	BOSTON REDEVELOPMENT AUTHORITY

IDENTIFICATION NUMBERS	TITLE HOLDERS
11 S	BOSTON REDEVELOPMENT AUTHORITY
12 S	COMMONWEALTH OF MASS - D PW
13 S	BOSTON REDEVELOPMENT AUTHORITY
14 S	COMMONWEALTH OF MASS
15 S	COMMONWEALTH OF MASS
16 S	BOSTON REDEVELOPMENT AUTHORITY
17 S	COMMONWEALTH OF MASS.
18 S	BOSTON REDEVELOPMENT AUTHORITY
19 S	Y M C A
20 S	Y M C A.

IDENTIFICATION NUMBERS	TITLE HOLDERS
21 S	COMMONWEALTH OF MASS - M D C
22 S	COMMONWEALTH OF MASS - M D C
23 S	COMMONWEALTH OF MASS
24 S	BOSTON EDISON CO
25 S	BOSTON EDISON CO
26 S	COMMONWEALTH OF MASS
27 S	COMMONWEALTH OF MASS.
28 S	COMMONWEALTH OF MASS - D P W
29 S	COMMONWEALTH OF MASS
30 S	ROBERT C. LINNELL

IDENTIFICATION NUMBERS	TITLE HOLDERS
31 S	M B T A
32 S	ROBERT C LINNELL
33 S	ROBERT C LINNELL
34 S	CITY OF BOSTON
35 S	GENERAL HOSPITAL CORP
36 S	JOSIAN A SPAULDING
37 S	B & M RAILROAD
38 S	COMMONWEALTH OF MASS

Figure 40
Preferred Alternative (5A Modified 4 Lane Tunnel) Property and Utilities Plan - North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

IDENTIFICATION NUMBERS	TITLE HOLDERS
102 S	THE GILLETTE CO
103 S	THE GILLETTE CO
104 S	THE GILLETTE CO
105 S	BOSTON WHARF CO
106 S	BOSTON WHARF CO.
107 S	BOSTON WHARF CO.
108 S	BOSTON WHARF CO
109 S	BOSTON WHARF CO.
110 S	BOSTON WHARF CO
111 S	WENDEEL A DUFFEY

IDENTIFICATION NUMBERS	TITLE HOLDERS
112 S	HUB OBL CORP
113 S	OILIB CORP
114 S	JOSEPH DIRICO
115 S	ARTHER SOLOMON
116 S	LEONARO M LEWIS
117 S	SLADE GORTON B CO
118 S	BOSTON WHARF CO
119 S	BOSTON WHARF CO
120 S	UNITEO POSTAL SERVICE
121 S	F.P.C. PROPERTIES INC

IDENTIFICATION NUMBERS	TITLE HOLDERS
122 S	JAMES S. SPINALE
123 S	ARABAN COFFEE CO. INC
124 S	ROBERT E PILL
125 S	F.P.C. PROPERTIES
126 S	NICHOLAS J CONTOS
127 S	NEW ENGLAND SEA FOOD CENTER INC
128 S	FAN REALTY CORP
129 S	STANDEX INTERNATIONAL CORP
130 S	WILLIAM C CASEY
131 S	MASS PORT AUTHORITY

IDENTIFICATION NUMBERS	TITLE HOLOERS
151 S	BIANCHI ASSOCIATES
152 S	HOUSE OF BIANCHI, INC
153 S	BONNIE REALTY TRUST
154 S	E.D.I.C. OF BOSTON

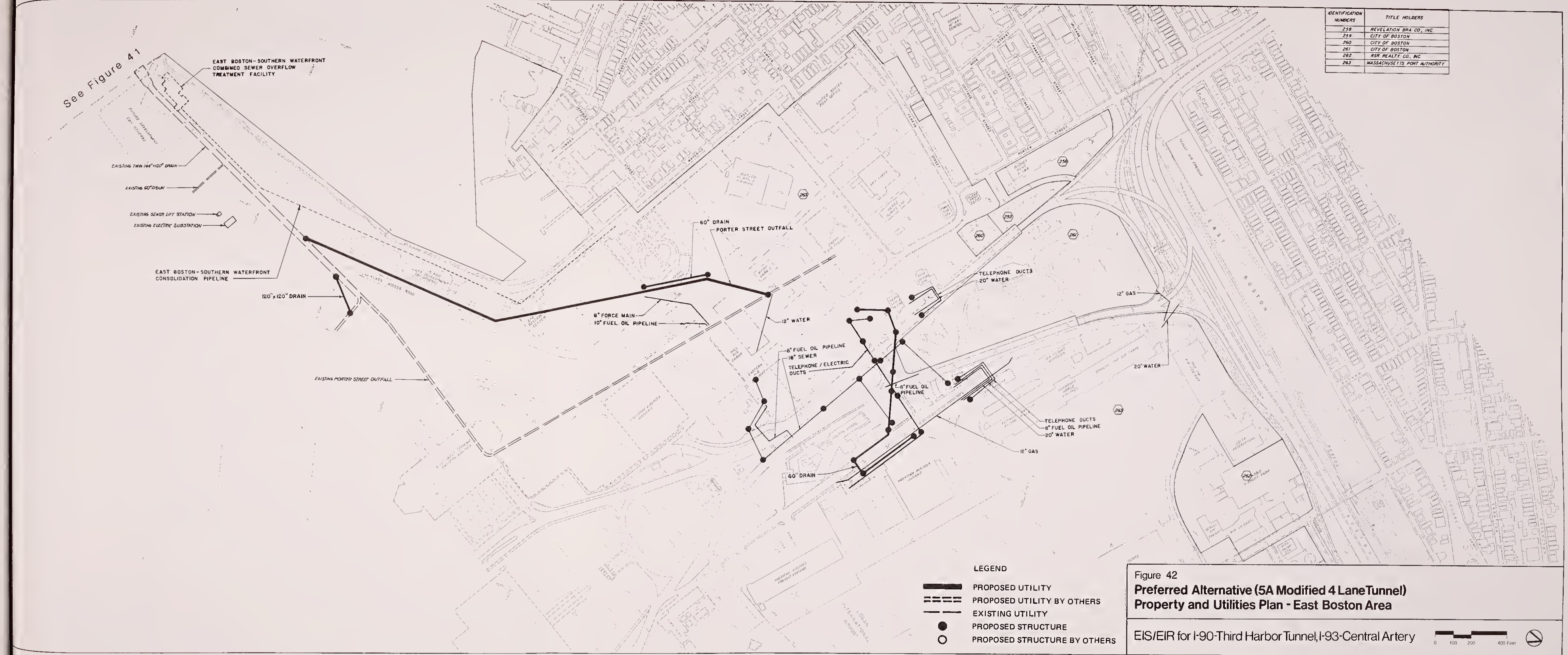
LEGEND

- PROPOSED UTILITY
- PROPOSED UTILITY BY OTHERS
- EXISTING UTILITY
- PROPOSED STRUCTURE
- PROPOSED STRUCTURE BY OTHERS

Figure 41
Preferred Alternative (5A Modified 4 Lane Tunnel)
Property and Utilities Plan - South Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

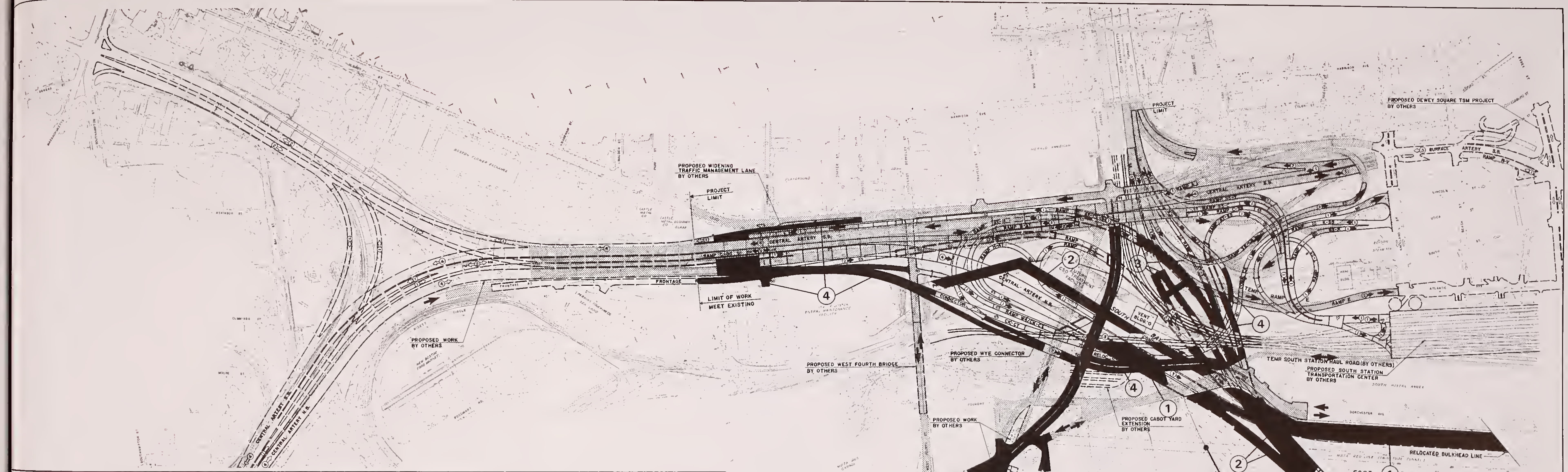
0 100 200 400 Feet



IDENTIFICATION NUMBERS	TITLE HOLDERS
258	REVELATION BRA CO, INC
259	CITY OF BOSTON
260	CITY OF BOSTON
261	CITY OF BOSTON
262	RSR REALTY CO., INC
263	MASSACHUSETTS PORT AUTHORITY

Figure 42
Preferred Alternative (5A Modified 4 Lane Tunnel) Property and Utilities Plan - East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



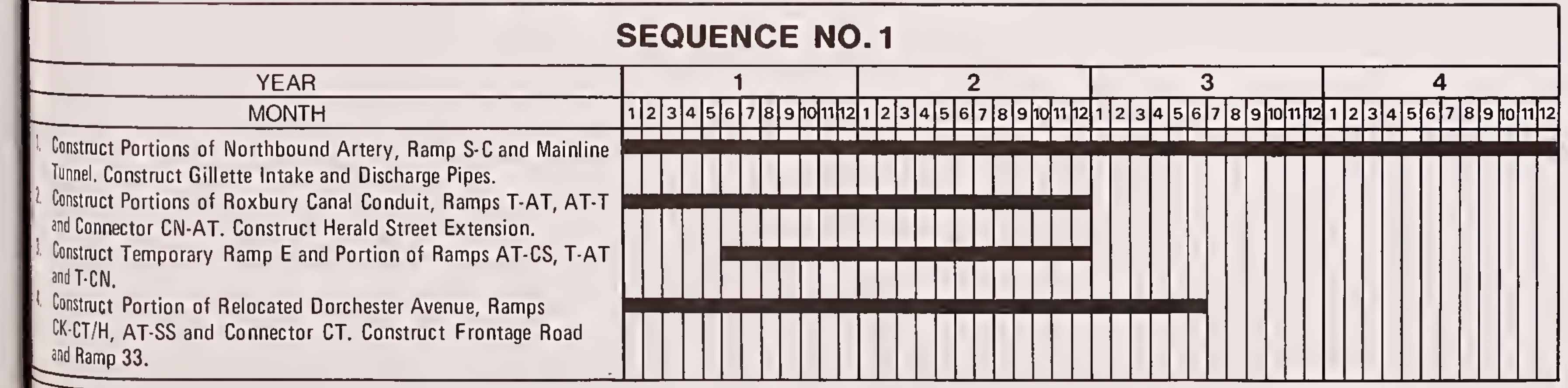
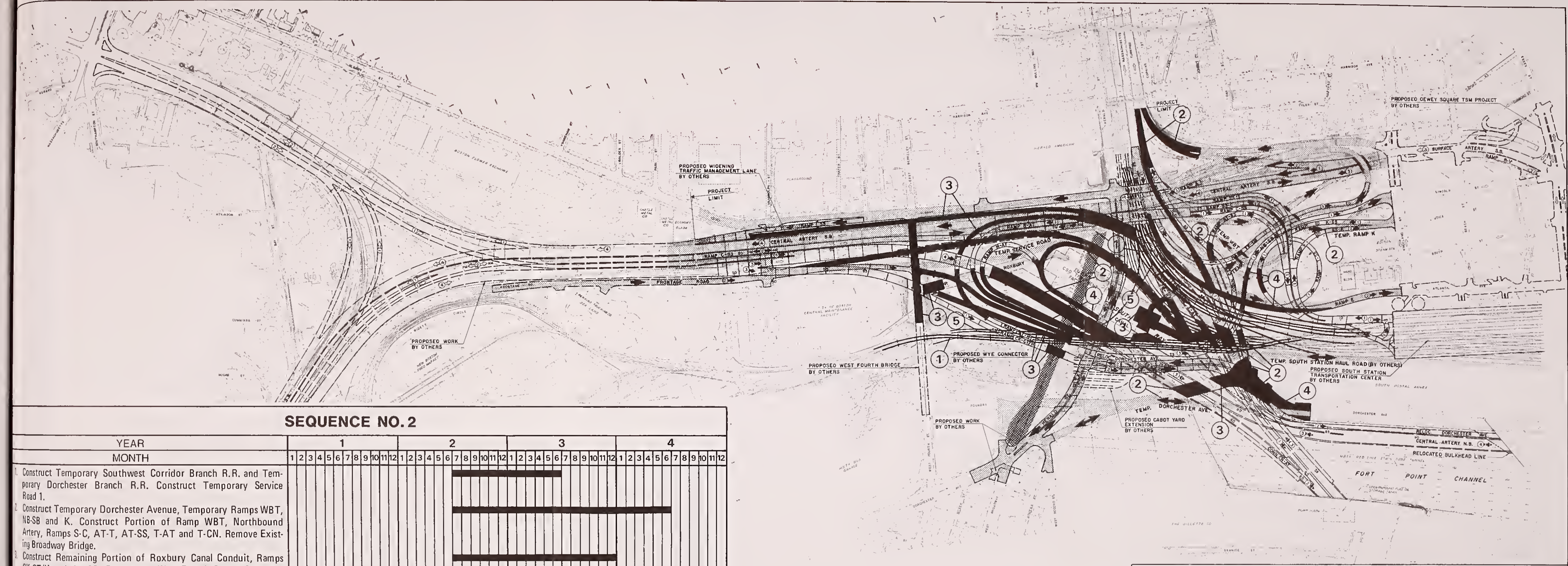


Figure 43
Preferred Alternative (5A Modified 4 Lane Tunnel)–Stage Construction South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



SEQUENCE NO. 2

YEAR	1												2												3												4											
MONTH	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
1. Construct Temporary Southwest Corridor Branch R.R. and Temporary Dorchester Branch R.R. Construct Temporary Service Road 1.																			■	■	■	■	■	■	■	■	■	■	■	■																		
2. Construct Temporary Dorchester Avenue, Temporary Ramps WBT, NB-SB and K. Construct Portion of Ramp WBT, Northbound Artery, Ramps S-C, AT-T, AT-SS, T-AT and T-CN. Remove Existing Broadway Bridge.																			■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■						
3. Construct Remaining Portion of Roxbury Canal Conduit, Ramps CK-CT/H and AT-CS. Reconstruct West Fourth Street. Construct Portion of Ramps AT-H, W4/CK-CT and CK-CT. Construct Vent Building No. 0.																			■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■												
4. Construct Portion of Relocated Dorchester Avenue, Ramp E, K-SS, H-AT and Connector CK.																									■	■	■	■	■	■	■	■	■	■	■	■	■	■										
5. Rebuild Southwest Corridor Branch R.R. to Original Location. Rebuild Dorchester Branch R.R. to Original Location.																																											■	■	■	■	■	■

LEGEND

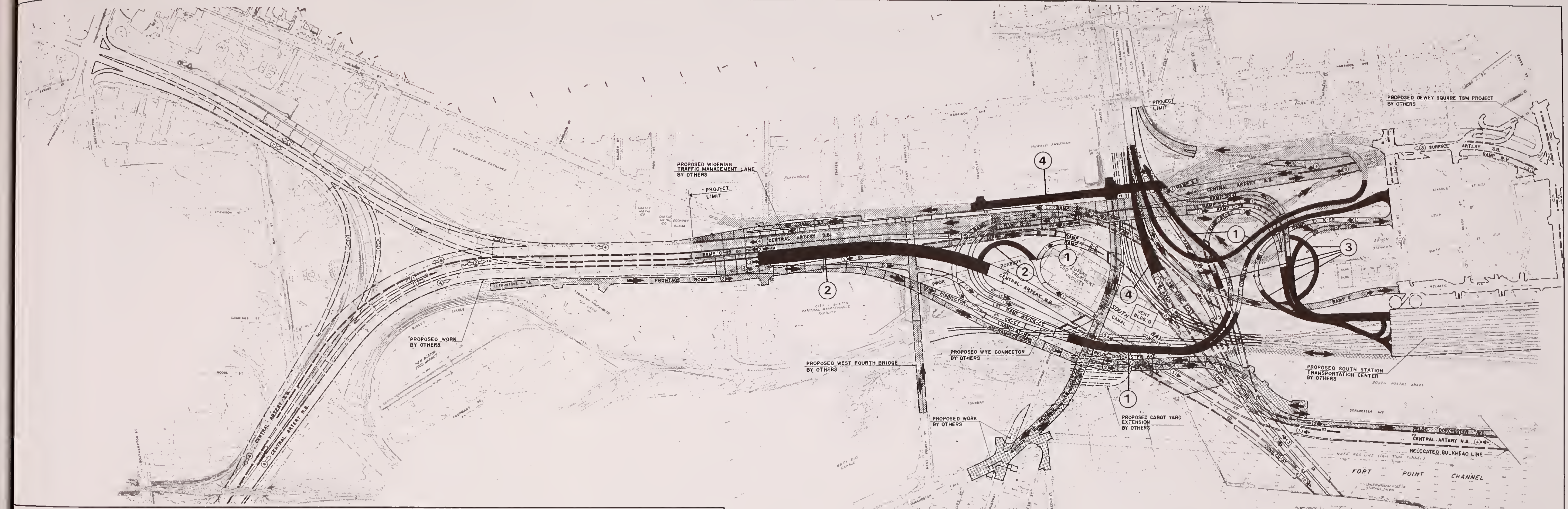
CONSTRUCTION PERFORMED DURING THIS PHASE

ROADWAYS IN SERVICE

ROADWAYS REMOVED

Figure 44
Preferred Alternative (5A Modified 4 Lane Tunnel)–Stage Construction South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



SEQUENCE NO. 3

YEAR	1												2												3												4											
MONTH	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
Construct Temp. Service Road No. 2. Construct Portions of Connectors CT, CK and Ramp TE-CS.																															■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■		
Construct Remaining Portion of Northbound Artery and Ramp H-AT.																																					■	■	■	■	■	■	■	■	■	■	■	■
Construct Portion of Ramps T-SS, AT-SS and SS-K.																													■	■	■	■	■	■	■	■	■	■	■	■	■	■						
Construct Albany Street and Remaining Portion of Ramp T-AT.																																					■	■	■	■	■	■	■	■				

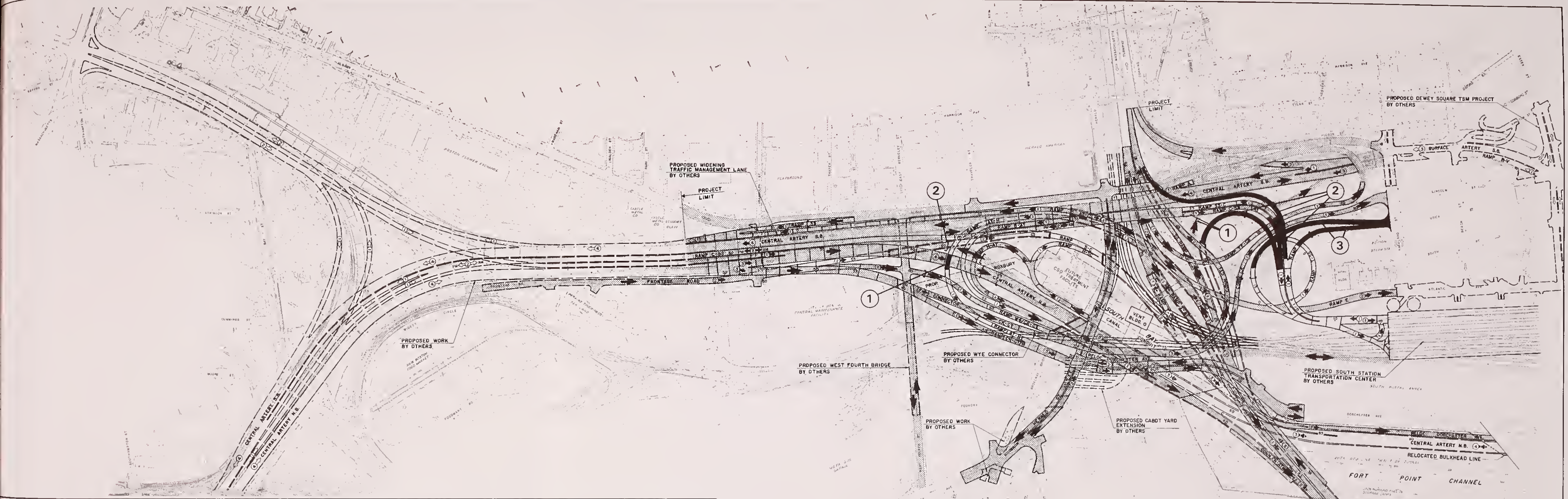
LEGEND

- CONSTRUCTION PERFORMED DURING THIS PHASE
- ROADWAYS IN SERVICE
- ROADWAYS REMOVED

Figure 45
Preferred Alternative (5A Modified 4 Lane Tunnel)–Stage Construction South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 100 200 400



SEQUENCE NO. 4

YEAR	1												2												3												4											
MONTH	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
1. Construct Temporary Ramps CT-NBA, Temporary N.B. Artery and Remaining Portion of Ramp AT-H.																																									■	■	■					
2. Construct Remaining Portion of Ramps SS-T and AT-SS. Construct Portion of Ramp C-SS. Construct Temporary Northbound Artery Haul Road to be in Use from the End of Year 4 to the End of Year 8.																																											■	■	■	■	■	■
3. Construct Remaining Portion of Ramps K-SS and SS-K.																																											■	■	■			

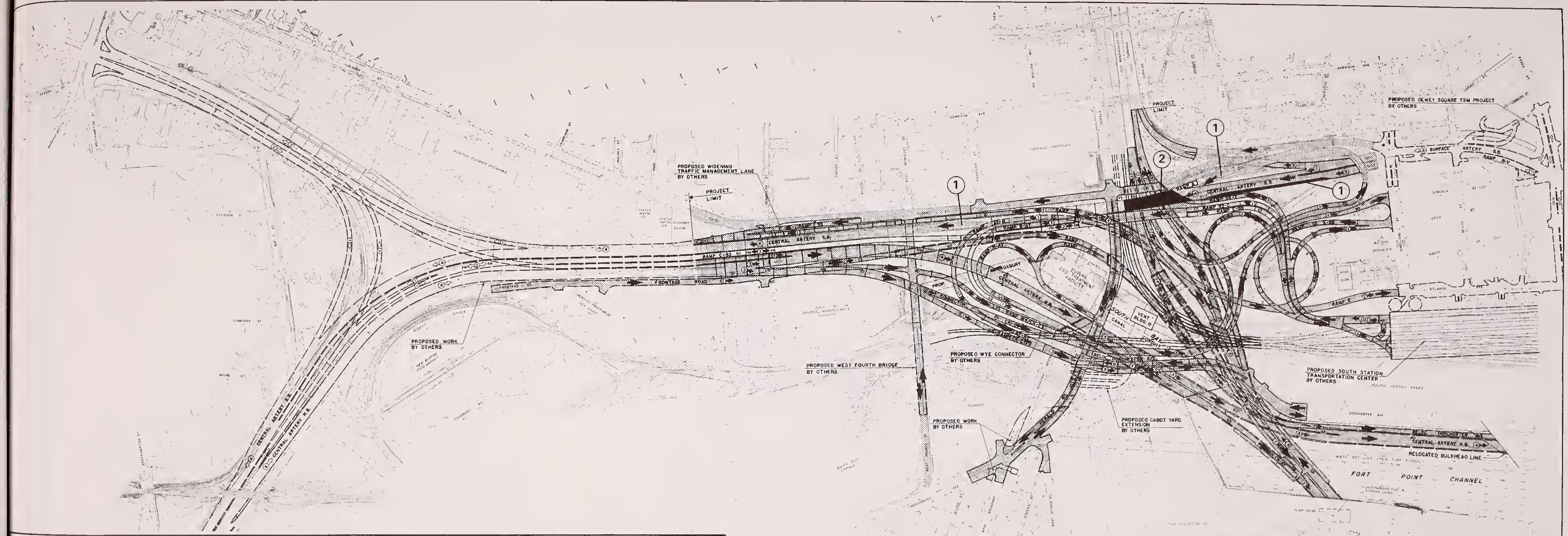
LEGEND

- CONSTRUCTION PERFORMED DURING THIS PHASE
- ROADWAYS IN SERVICE
- ROADWAYS REMOVED

Figure 46
Preferred Alternative (5A Modified 4 Lane Tunnel)–Stage Construction South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 100 200 400



SEQUENCE NO. 5

YEAR	5,6 & 7	8	9	10
MONTH	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12
1. Construct Temporary Alignment for Southbound Artery Traffic. Widen Northbound Artery for Southbound Traffic. Construct Temporary Ramp A.				
2. Reconstruct Southbound Artery.				

LEGEND

CONSTRUCTION PERFORMED DURING THIS PHASE

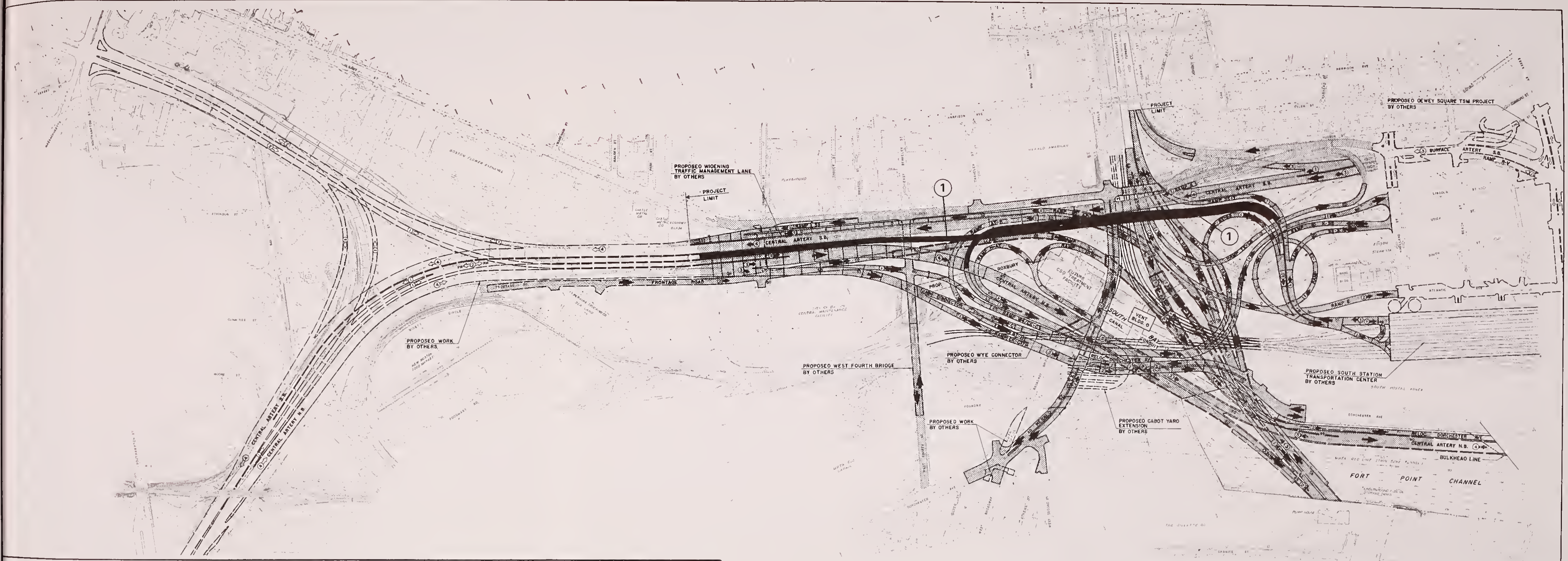
ROADWAYS IN SERVICE

ROADWAYS REMOVED

Figure 47
Preferred Alternative (5A Modified 4 Lane Tunnel)–Stage Construction South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 100 200 400



SEQUENCE NO. 6

YEAR	8	9	10	11
MONTH	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12
1. Construct Ramps C-SS, B-AT and SS-C.				

LEGEND

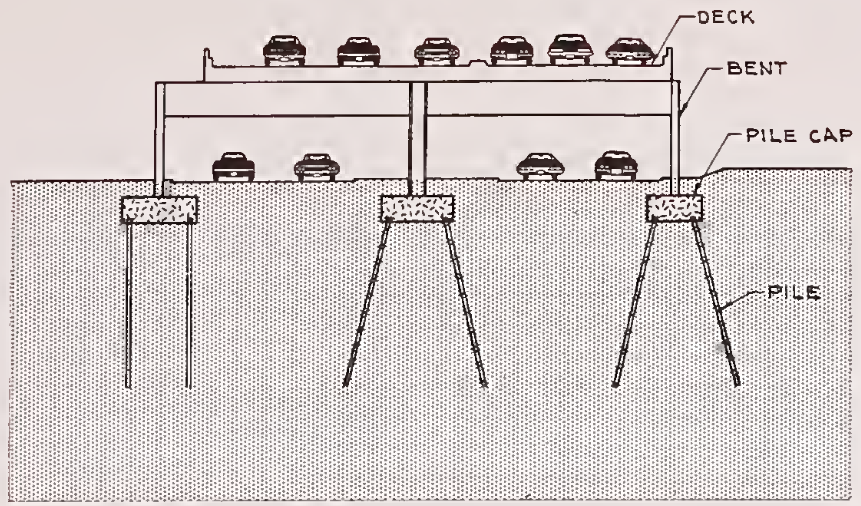
- CONSTRUCTION PERFORMED DURING THIS PHASE
- ROADWAYS IN SERVICE
- ROADWAYS REMOVED

Figure 4.8

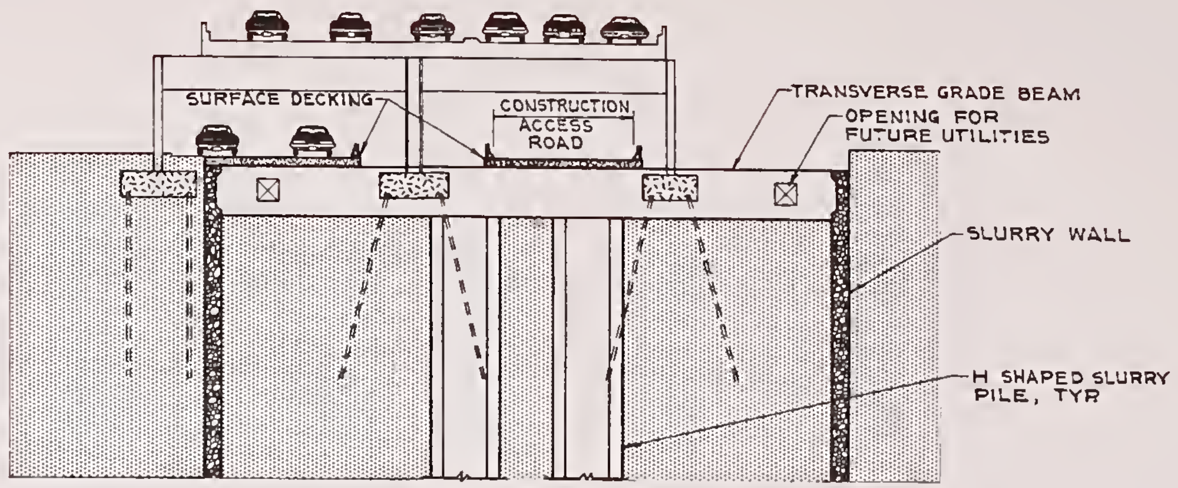
Preferred Alternative (5A Modified 4 Lane Tunnel)–Stage Construction South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

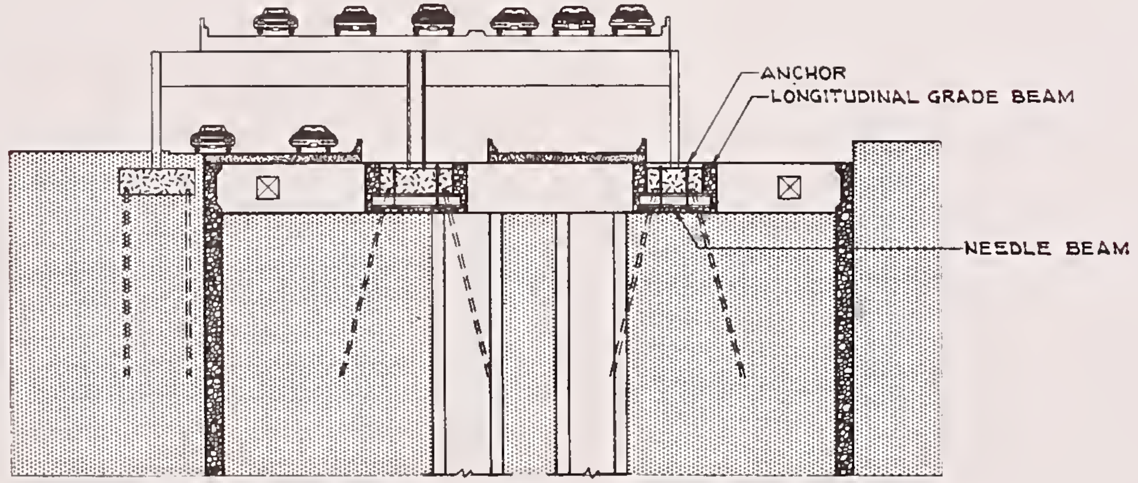
0 100 200 400



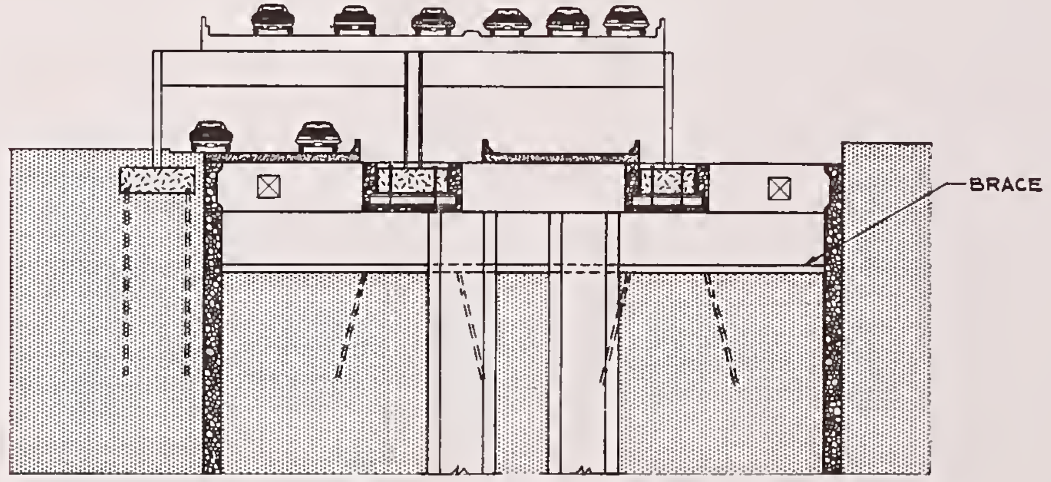
1. EXISTING CONDITION



2. SLURRY WALL - SLURRY PILES - TRANSVERSE GRADE BEAM - SURFACE DECKING



3. NEEDLE BEAM - LONGITUDINAL GRADE BEAM - ANCHORS



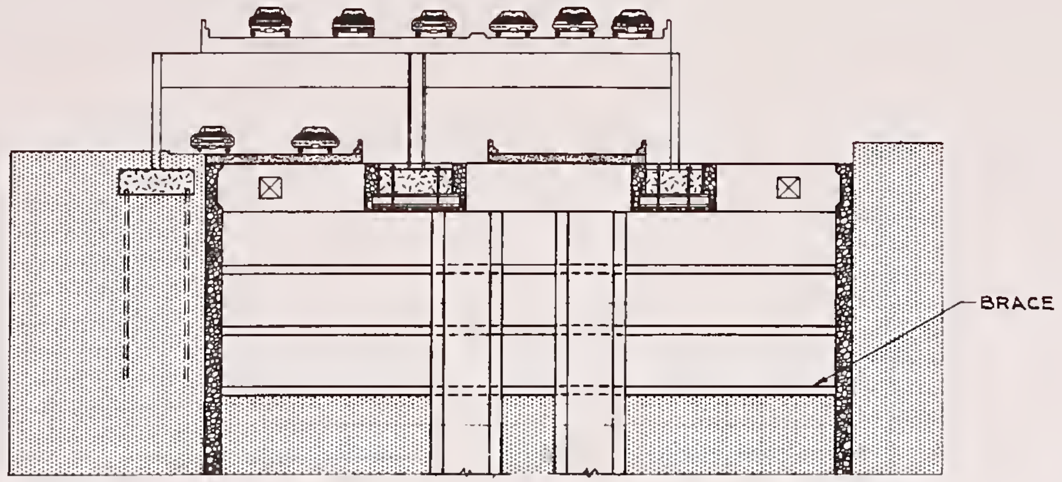
4. EXCAVATE - PILE REMOVAL - INTERMEDIATE BRACING

Figure 49

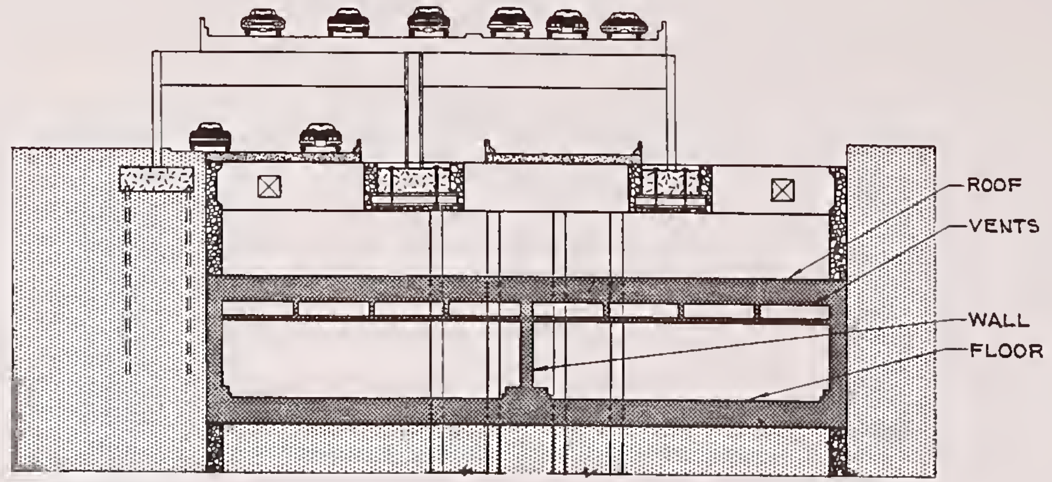
Preferred Alternative (5A Modified 4 Lane Tunnel)
Central Area - Stage Construction Typical Sections

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

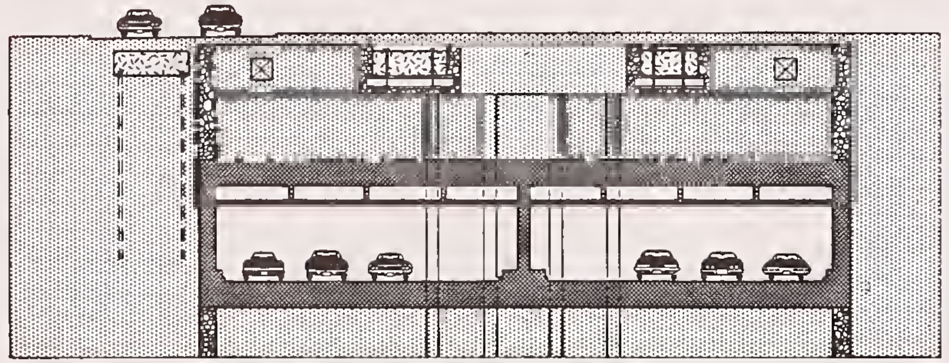
0 5 10 20 Feet



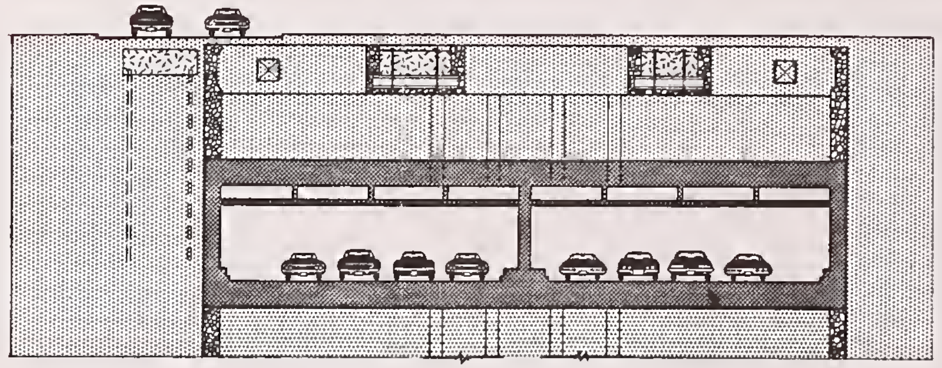
5. EXCAVATE - PILE REMOVAL - BOTTOM BRACING



6. CENTRAL ARTERY TUNNEL CONSTRUCTION



7. BACKFILL - REMOVE ELEVATED CENTRAL ARTERY - SURFACE GRADING



8. REMOVE INTERMEDIATE SLURRY PILE

Figure 50
Preferred Alternative (5A Modified 4 Lane Tunnel)
Central Area – Stage Construction Typical Sections

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 5 10 20 Feet

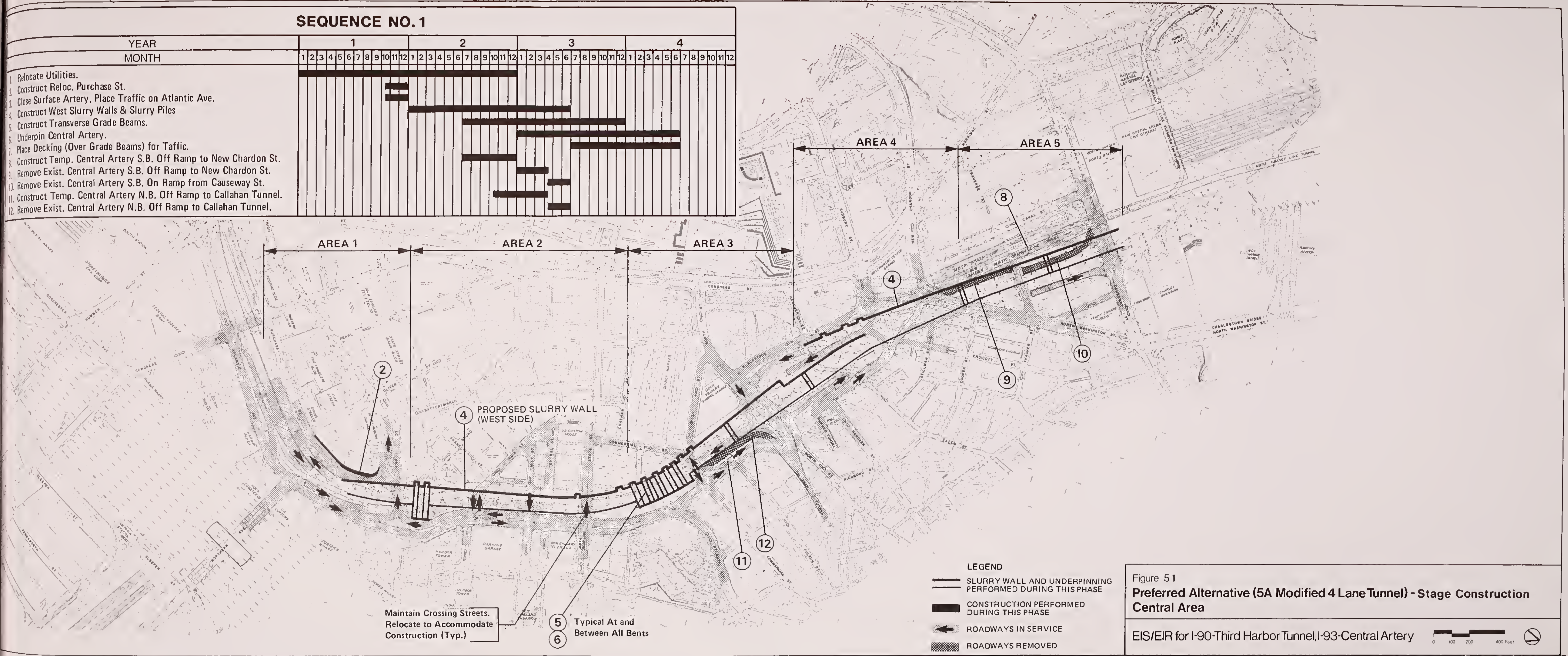


Figure 51

Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

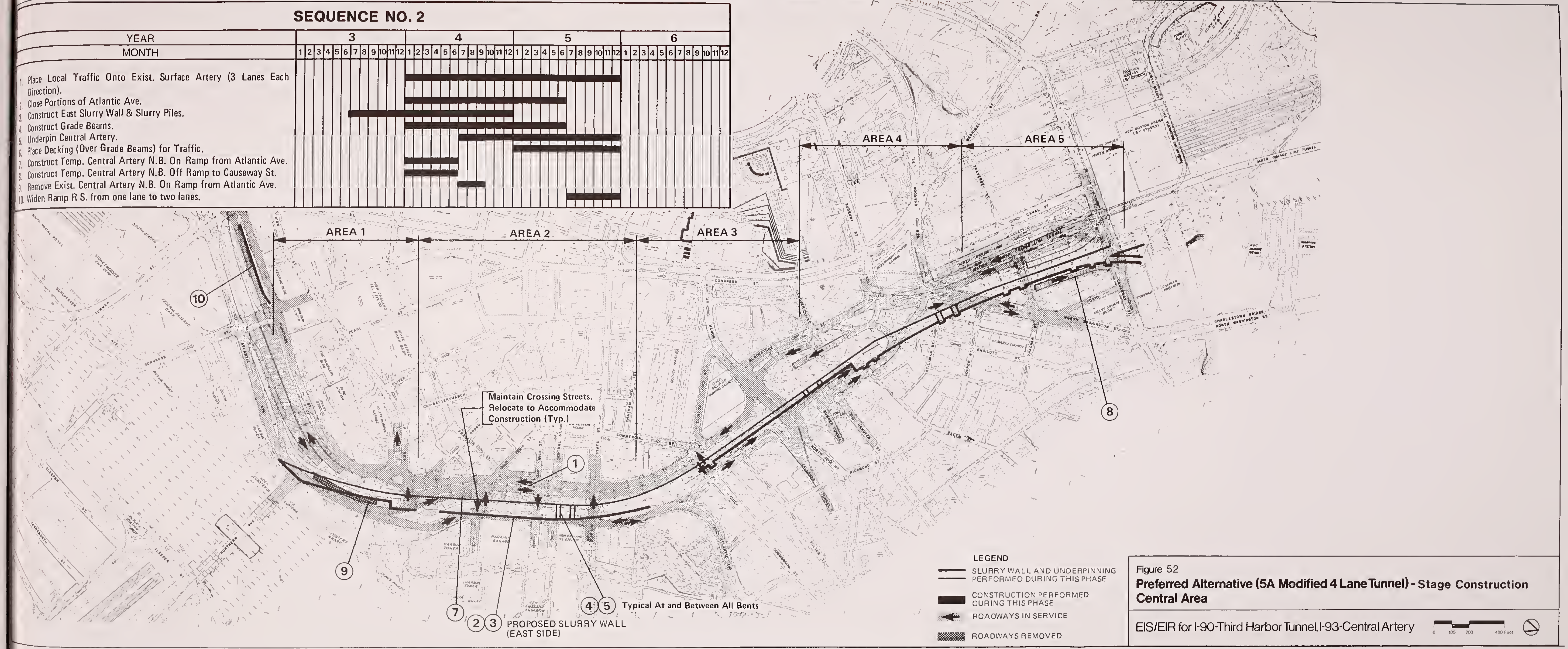


Figure 52
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

SEQUENCE NO. 3

	Year 5 (Month)	Year 6 (Month)	Year 7 (Month)	Year 8 (Month)
1. Place Local Traffic Onto Existing Atlantic Ave. & Surface Artery S.B.		1–12	1–12	1–12
2. Establish Surface Artery N.B. as a Construction Haul Road.		1–12	1–12	1–12
3. Excavation for Tunnel.		1–12	1–12	
4. Tunnel Construction.			1–12	1–12
5. Construct Ventilation Buildings.			1–12	1–12
6. Construct Temporary Central Artery S.B. Off Ramp to North St. & Callahan Tunnel.		1–6		
7. Remove Existing Central Artery S.B. Off Ramp.		7–9		
8. Construct Portion of New Ramp CS-CT.		10–12	1–3	
9. Construct Temporary Central Artery N.B. On Ramp from Sumner Tunnel.			1–6	
10. Remove Existing Central Artery N.B. On Ramp from Sumner Tunnel.			7–9	
11. Construct Portion of New Ramp ST-CN.			10–12	1–4

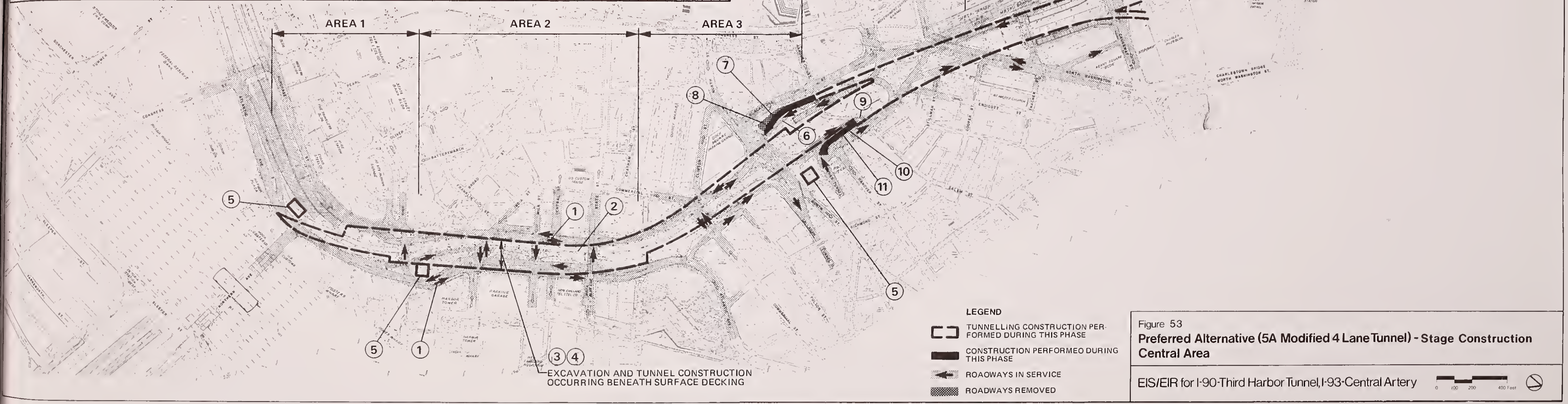
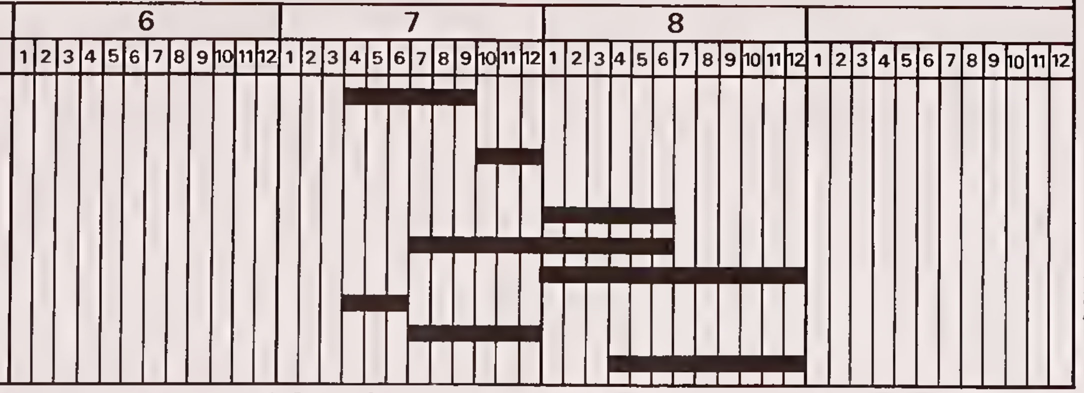


Figure 53
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

SEQUENCE NO. 4



1. Reconstruct Central Artery S.B. Off Ramp to North St. & Callahan Tunnel in Original Location.
2. Remove Temporary Central Artery S.B. Off Ramp to North St. & Callahan Tunnel.
3. Construct Portion of New Ramp CS-CT.
4. Construct Temp. Central Artery S.B. from High St. to Dewey Sq.
5. Construct Ramp A-CN & Portion of Ramp R-T.
6. Remove Temp. Central Artery N.B. Off Ramp to Callahan Tunnel.
7. Construct Ramp CN-SA.
8. Sheet-off and Construct One-half of Sumner Tunnel Approach.

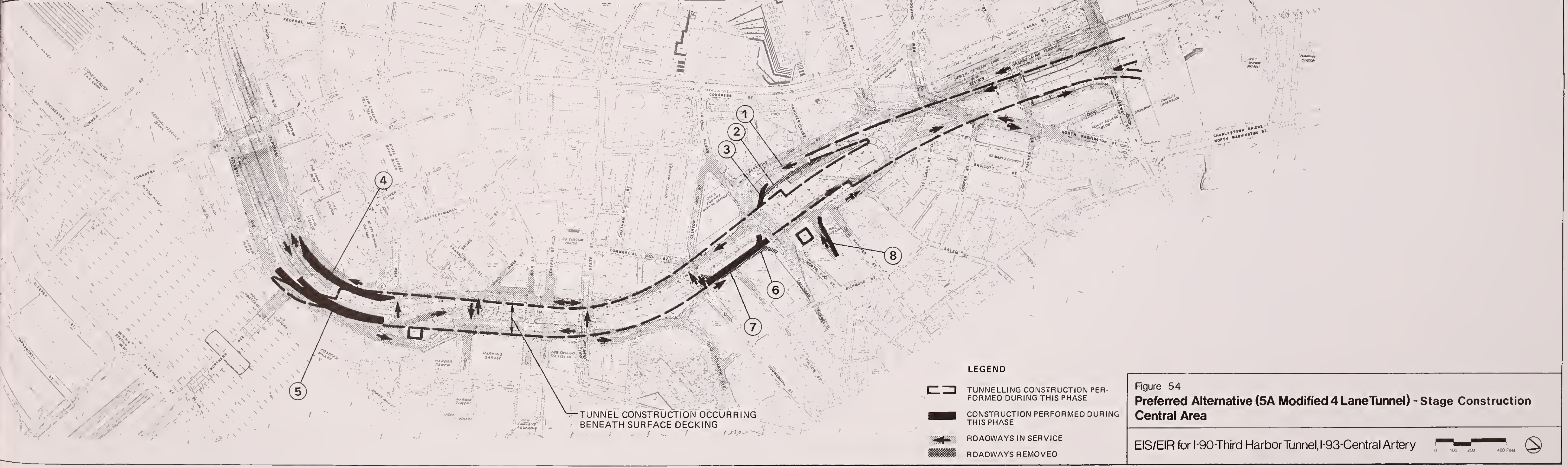


Figure 54
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Figure 55
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

SEQUENCE NO. 6

YEAR: 9, 10 — MONTH: 1 2 3 4 5 6 7 8 9 10 11 12 (for each year)

1. Place Traffic in Depressed Central Artery Southbound.
2. Maintain Existing Elevated Artery to North St. & High St. Off Ramps.
3. Remove Temporary Central Artery at High St. to Dewey Sq.
4. Dismantle Existing Elevated Artery from Bent No. 61 to Bent No. 65.
5. Construct Northern Half of Depressed Artery S.B. at Dewey Sq. & Ramp CS-P.
6. Construct "Cross Streets" at Oliver & Pearl St.
7. Dismantle Existing Elevated Artery from Bent No. 29 to Bent No. 33.
8. Remove Temp. Central Artery S.B. Off Ramp to New Chardon St.
9. Construct Detour for Sumner Tunnel to Cross St. Traffic.
10. Remove Temp. Central Artery S.B. On Ramp from Sudbury St.

Figure 56
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

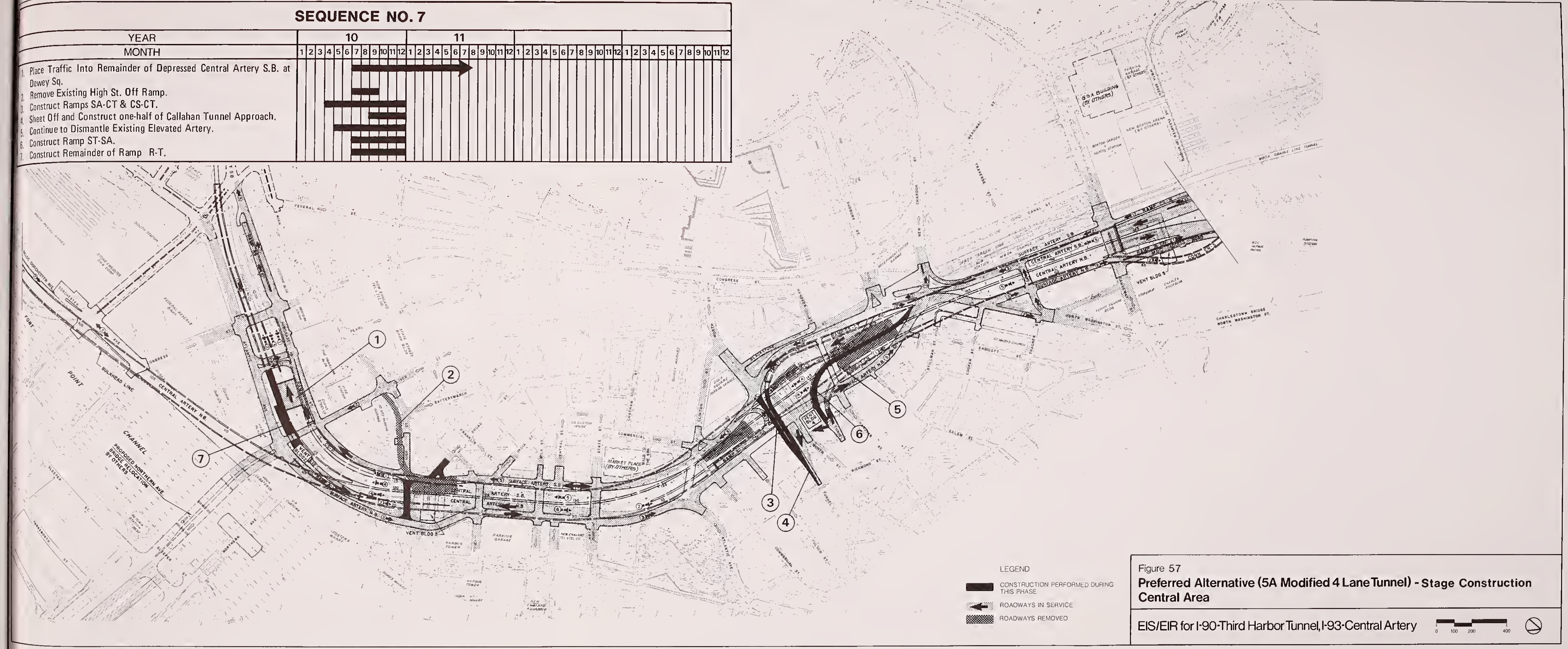


Figure 57
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

SEQUENCE NO. 8

YEAR	10	11		
MONTH	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12
1. Place Traffic Into New Ramps CS-CT & SA-CT.		▬▬▶		
2. Construct Remaining Half of Callahan Tunnel Approach.		▬▬		
3. Remove Temp. Central Artery S.B. Off Ramp to North St.		▬		
4. Place Traffic Into New Ramp ST-SA.		▬▶		
5. Construct Remaining Half of Sumner Tunnel Approach.		▬		
6. Continue Dismantling Existing Elevated Artery.		▬▬▬		
7. Construct Portions of Surface Artery N.B. & S.B.		▬▬▬		

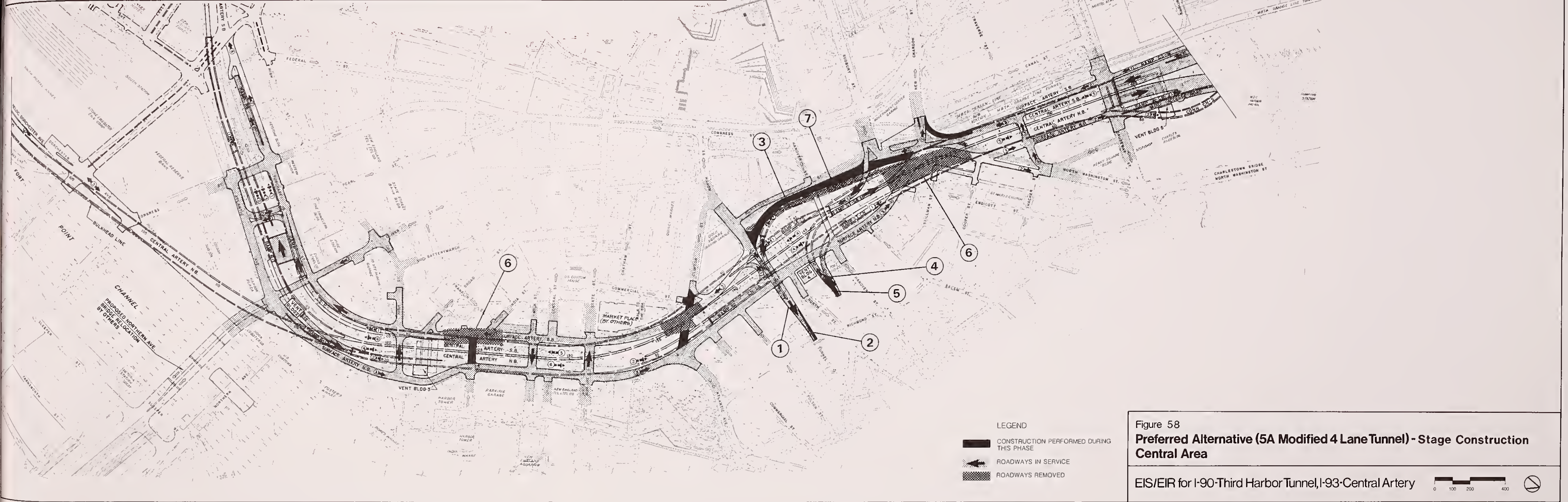


Figure 58
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

SEQUENCE NO. 9

YEAR	11	12		
MONTH	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12

1. Open All Ramps & Connections To & From Depressed Central Artery.
2. Dismantle Remaining Portions of Elevated Central Artery.
3. Remove Intermediate Slurry Piles in the Depressed Tunnel.
4. Complete Surface Artery N.B. & S.B.

LEGEND

CONSTRUCTION PERFORMED DURING THIS PHASE

ROADWAYS IN SERVICE

ROADWAYS REMOVED

Figure 59
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction Central Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 100 200 400

SEQUENCE NO. 1

YEAR	5												6											
MONTH	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
1. Construct Route I-93 S.B. Trusses.	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■						
2. Remove Charles River Building.*	■	■	■	■	■	■																		
3. Modify Ramps from "North Area Project."	■	■	■	■	■	■																		
4. Construct Connectors CN-S and S-CS and Ramp CN-L Up to R.R. Tracks.	■	■	■	■	■	■	■	■	■	■	■	■												
5. Construct I-93 N.B. From River to Sta. 194+00.							■	■	■	■	■	■	■	■	■	■	■	■						
6. Construct Vent Building No. 5.							■	■	■	■	■	■	■	■	■	■	■	■	■	■	■			
7. Remove Anelex Building.*	■	■	■	■																				
8. Construct Connector CN-S Vent Building to Sta. 71+50.							■	■	■	■	■	■												

*Note: These Buildings may be Removed in Year 1.

Figure 60
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 100 200 400 Feet

SEQUENCE NO. 2

YEAR	6												7											
MONTH	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
1. Construct Route I-93 N.B. Trusses.							■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■
2. Construct Seven Sets of Temporary R.R. Tracks.	■	■	■	■																				
3. Transfer Train Movements and Remove Tracks 1 through 7.					■	■																		
4. Construct Connectors CN-S and S-CS Up to Track 8.							■	■	■	■	■	■												
5. Construct Route I-93 S.B. From River to Sta. 190+00.							■	■	■	■	■	■	■	■	■	■	■	■						
6. Construct Connector CN-S From Vent Building to Causeway Street.													■	■	■	■	■	■						
7. Construct MDC Access Road.																	■	■	■	■				

Figure 6 1
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

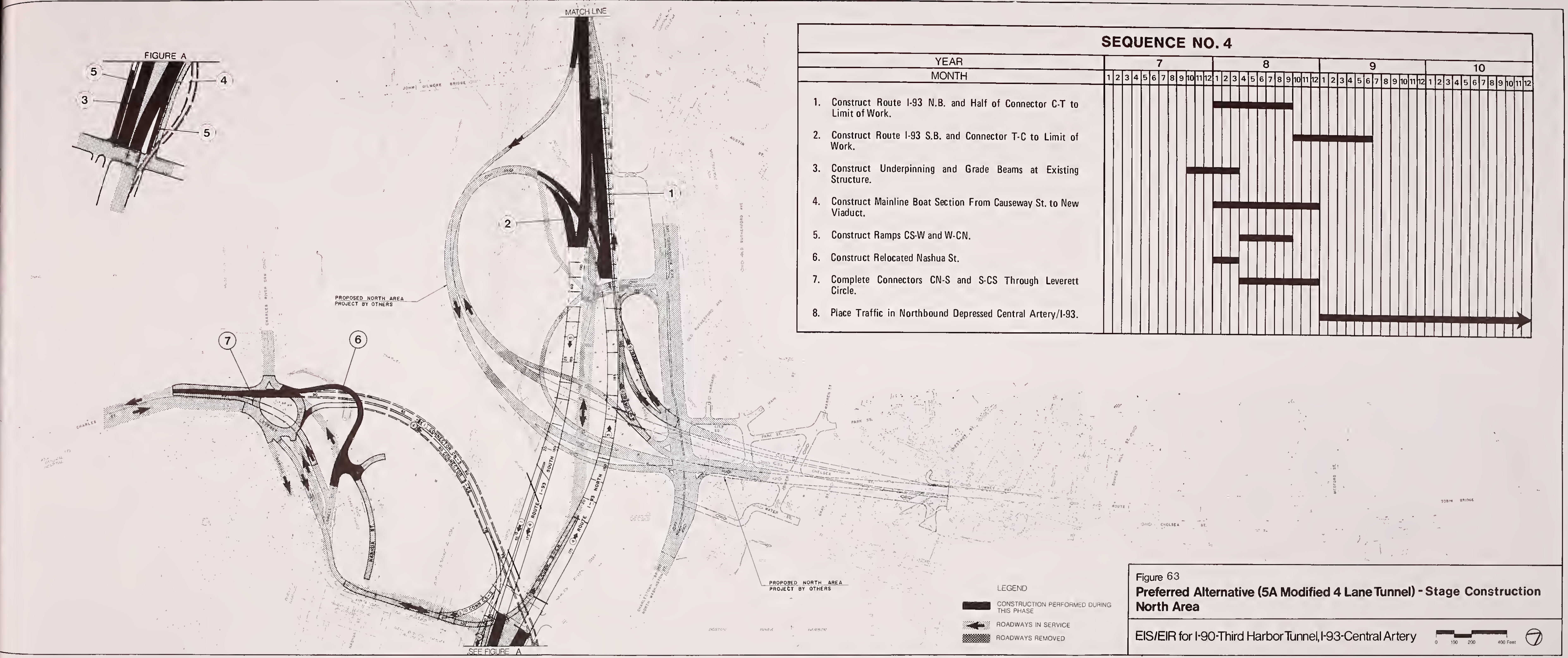
0 100 200 400 Feet

SEQUENCE NO. 3

YEAR	6												7											
MONTH	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
1. Replace Tracks 1 Through 7 and Transfer Train Movements.													■	■	■									
2. Remove Tracks 8, 9, 10 and 4 Temporary Tracks.																■	■							
3. Construct Connectors CN-S and S-CS (Sta. 71+50 to Track 8)																		■	■	■	■	■	■	■
4. Construct Route I-93 S.B. From Abutment to New Truss.																■	■	■	■	■	■			
5. Construct Route I-93 N.B. From Abutment to New Truss.																			■	■	■	■	■	■
6. Construct Connector CS-S From Existing Connectors to New Truss.																			■	■	■	■		
7. Construct Ramp CS-W From Abutment to New Truss.																					■	■	■	■
8. Build Temporary Connector T-C.																			■	■	■	■	■	■

Figure 62
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

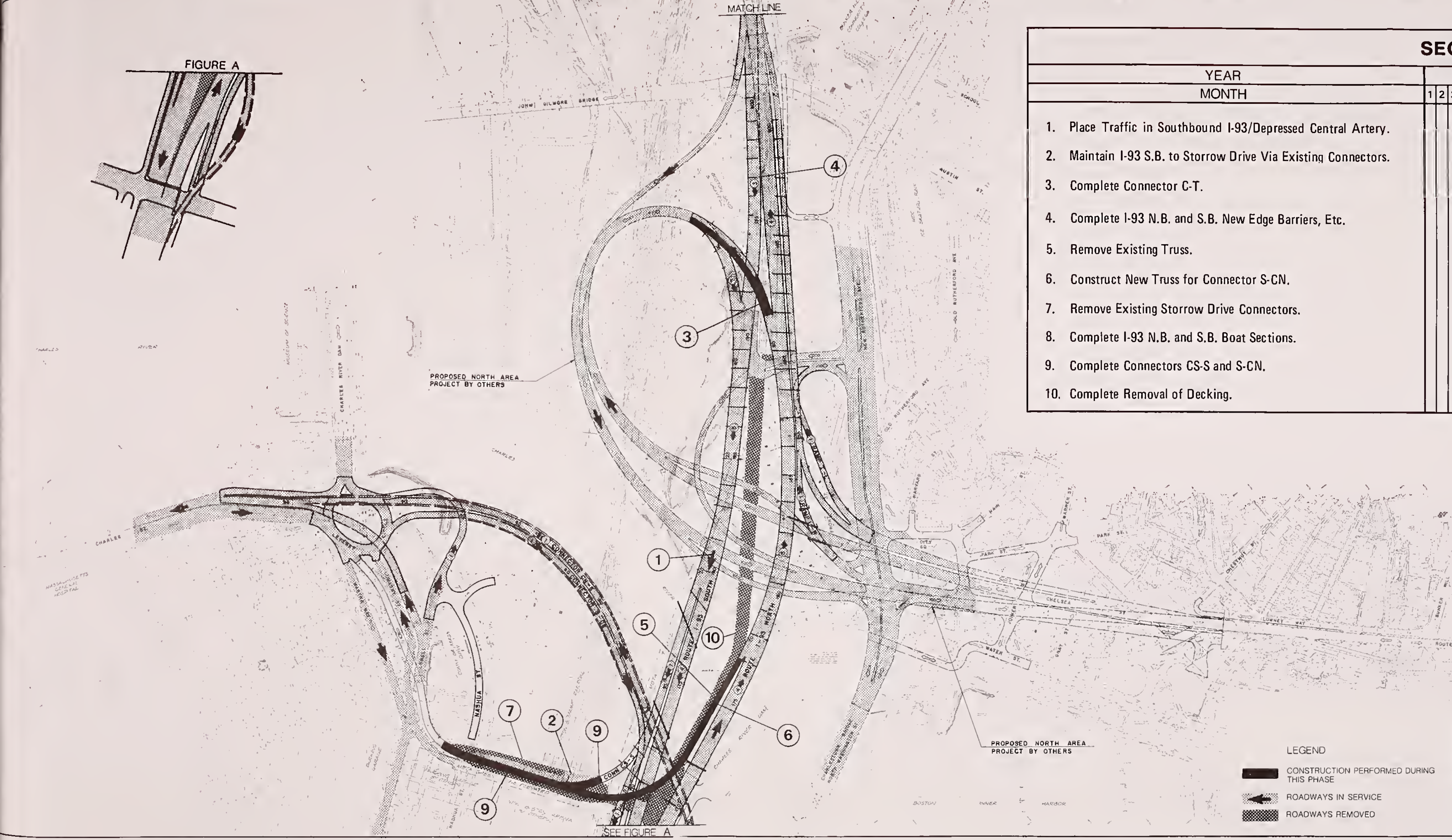


SEQUENCE NO. 4

YEAR	7	8	9	10
MONTH	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12
1. Construct Route I-93 N.B. and Half of Connector C-T to Limit of Work.				
2. Construct Route I-93 S.B. and Connector T-C to Limit of Work.				
3. Construct Underpinning and Grade Beams at Existing Structure.				
4. Construct Mainline Boat Section From Causeway St. to New Viaduct.				
5. Construct Ramps CS-W and W-CN.				
6. Construct Relocated Nashua St.				
7. Complete Connectors CN-S and S-CS Through Leverett Circle.				
8. Place Traffic in Northbound Depressed Central Artery/I-93.				

Figure 63
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



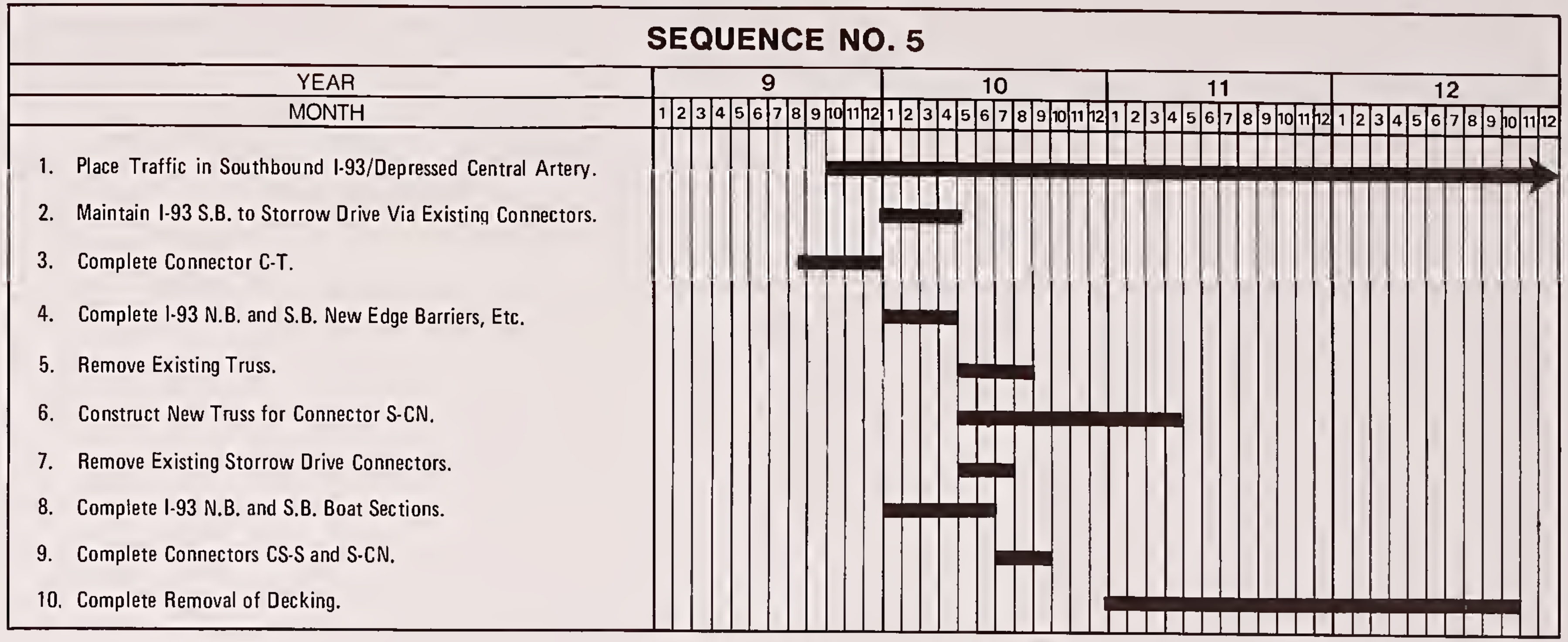


Figure 64

Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction North Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

SEQUENCE NO. 1

YEAR	1	2	3	4
MONTH	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12
1. Build Temporary Summer Street Bridge.				
2. Construct Temporary Detour Roads.				
3. Remove Existing Bridges.				
4. Relocate Conrail Track.				
5. Construct Mainline Tunnel, Ramps and Service Road.				
6. Construct Mainline Harbor Tunnel.				
7. Construct Vent Buildings No. 7 and 9.				
8. Construct Administration Building.				

LEGEND

CONSTRUCTION PERFORMED DURING THIS PHASE

ROADWAYS IN SERVICE

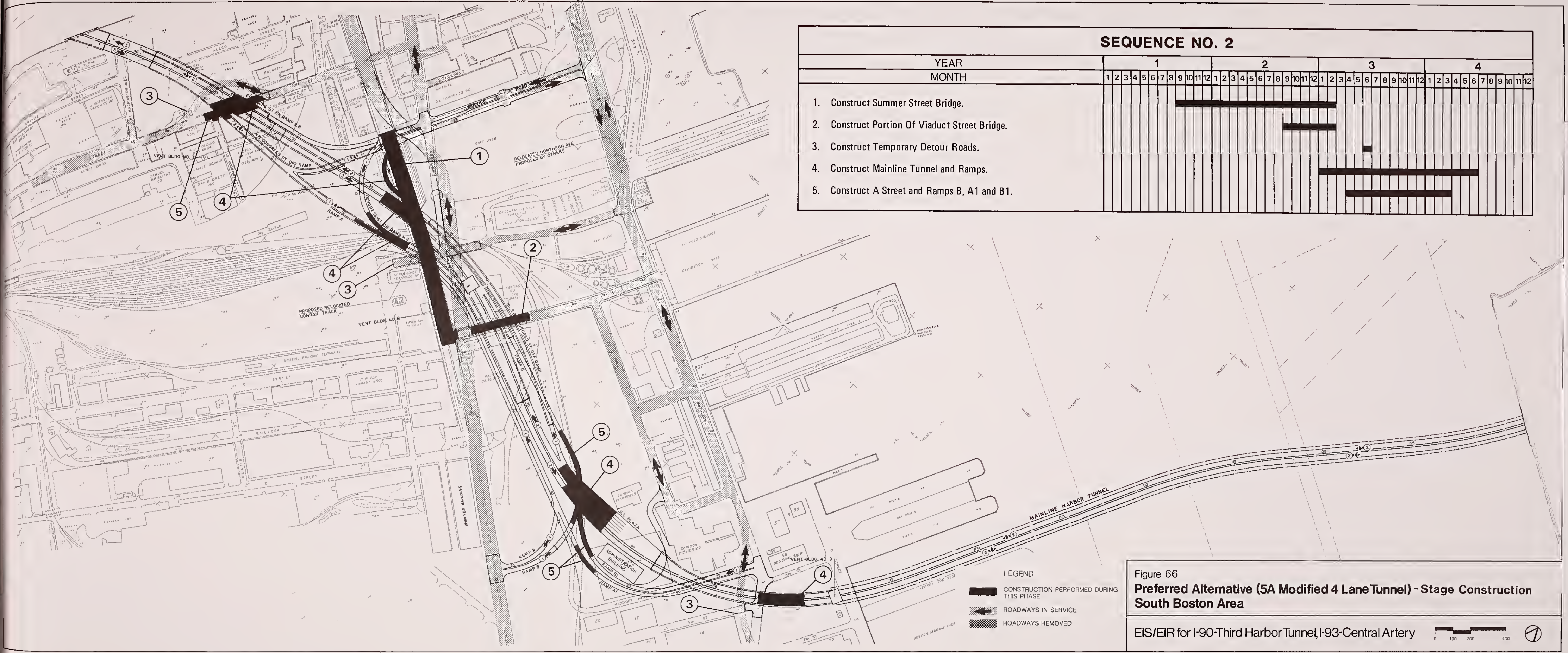
ROADWAYS REMOVED

Figure 65

Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction South Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 100 200 400



SEQUENCE NO. 2

YEAR	1	2	3	4
MONTH	1–12	1–12	1–12	1–12

1. Construct Summer Street Bridge.
2. Construct Portion Of Viaduct Street Bridge.
3. Construct Temporary Detour Roads.
4. Construct Mainline Tunnel and Ramps.
5. Construct A Street and Ramps B, A1 and B1.

LEGEND

CONSTRUCTION PERFORMED DURING THIS PHASE

ROADWAYS IN SERVICE

ROADWAYS REMOVED

Figure 66

Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction South Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

SEQUENCE NO. 3

1. Construct B Street Detour.
2. Remove Summer Street Detour.
3. Construct Remainder of Mainline Tunnel and Ramps.
4. Construct Remainder Of Viaduct Street Bridge.
5. Construct B Street, Congress Street and Northern Avenue.
6. Construct Vent Building No. 8.

Figure 67
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction South Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

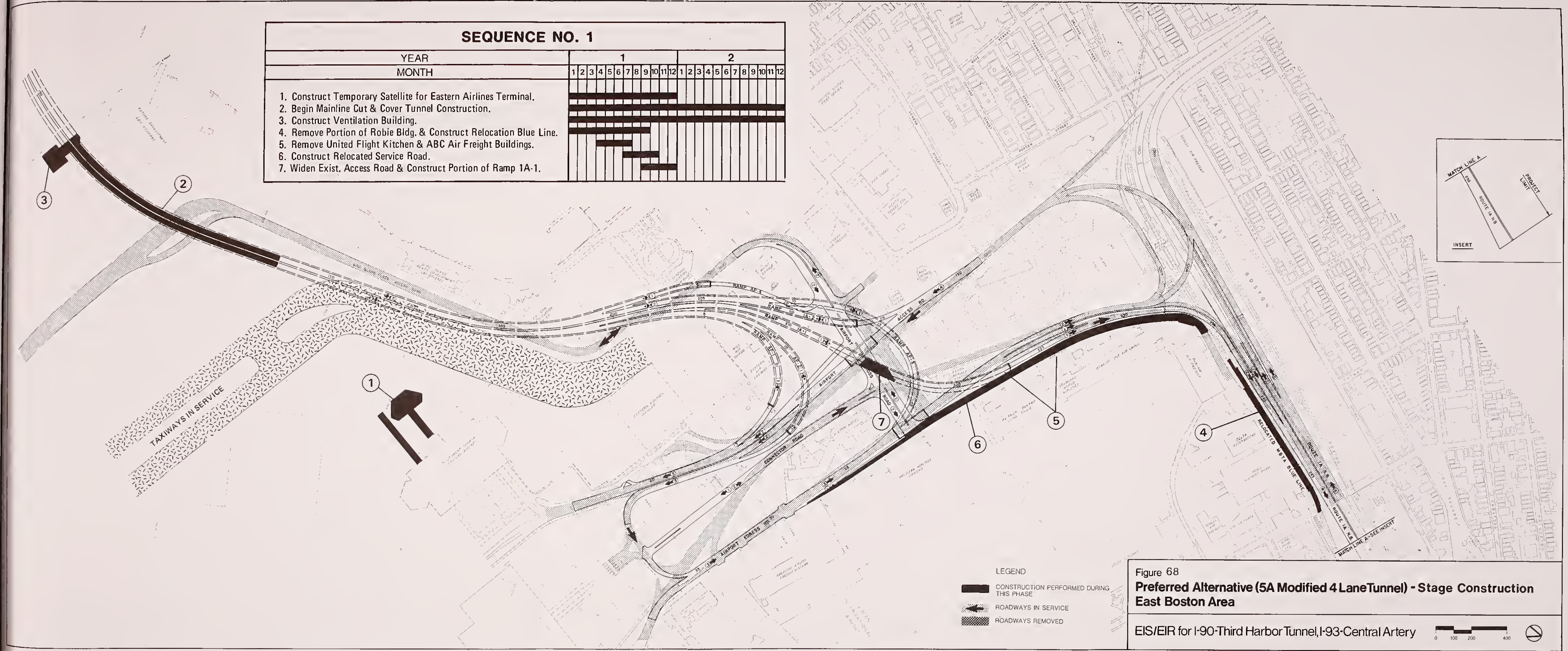


Figure 68
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

SEQUENCE NO. 2

YEAR	2												3											
MONTH	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
1. Transfer Shuttle to Temporary Satellite and Redefine Taxiway.	■																							
2. Relocate Eastern Res. Center and Continue Mainline Tunnel Const.	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■
3. Construct Temporary Airport Access Road.	■	■	■																					
4. Relocate Eastern Air Cargo, Hill Air Cargo & National Buildings.		■	■	■																				
5. Construct Portion of Ramps AP-1, AP-2, AP-3, 1A-1 and 1A-2.				■	■	■	■	■	■	■	■	■												
6. Remove Exxon Station and Construct Portion of Ramp 1A-1							■	■	■	■	■	■												
7. Construct Portion of Airport Egress Road & Widen Route 1A NB.	■	■	■	■																				

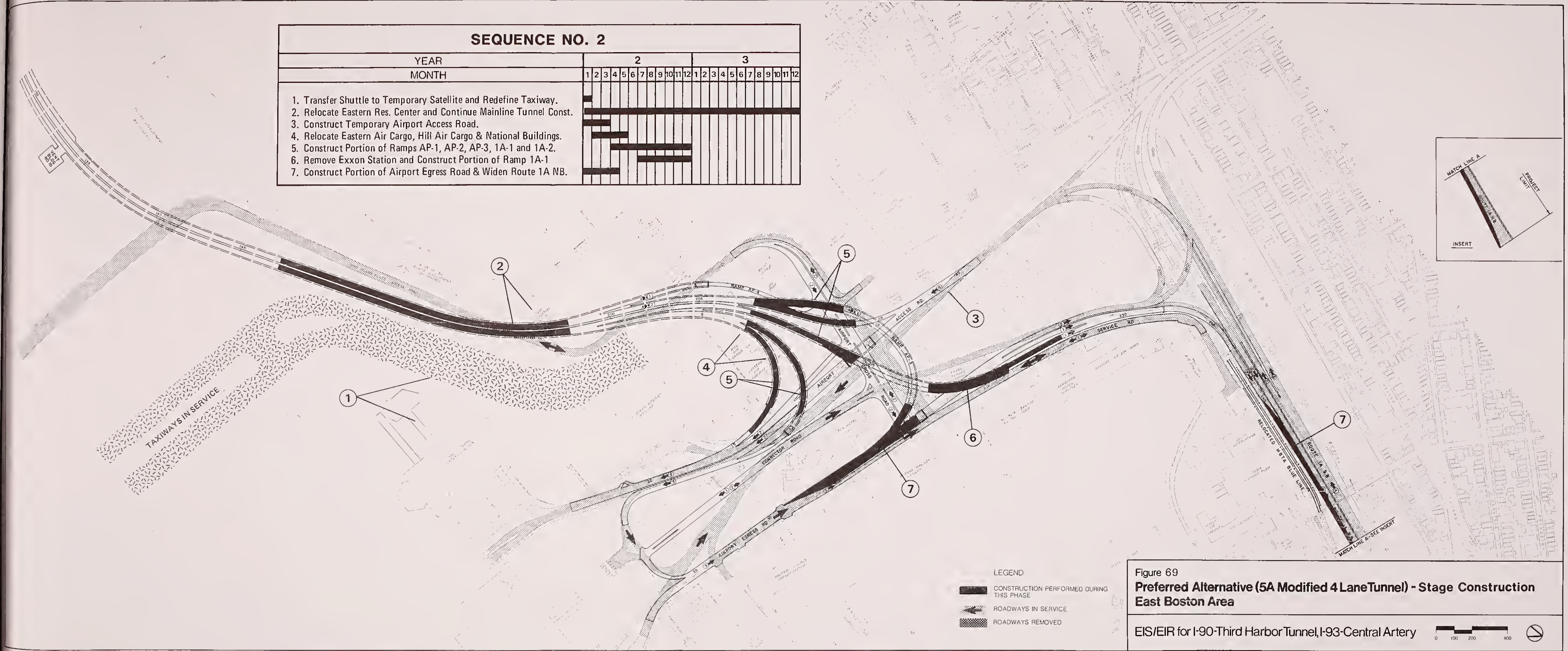


Figure 69
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

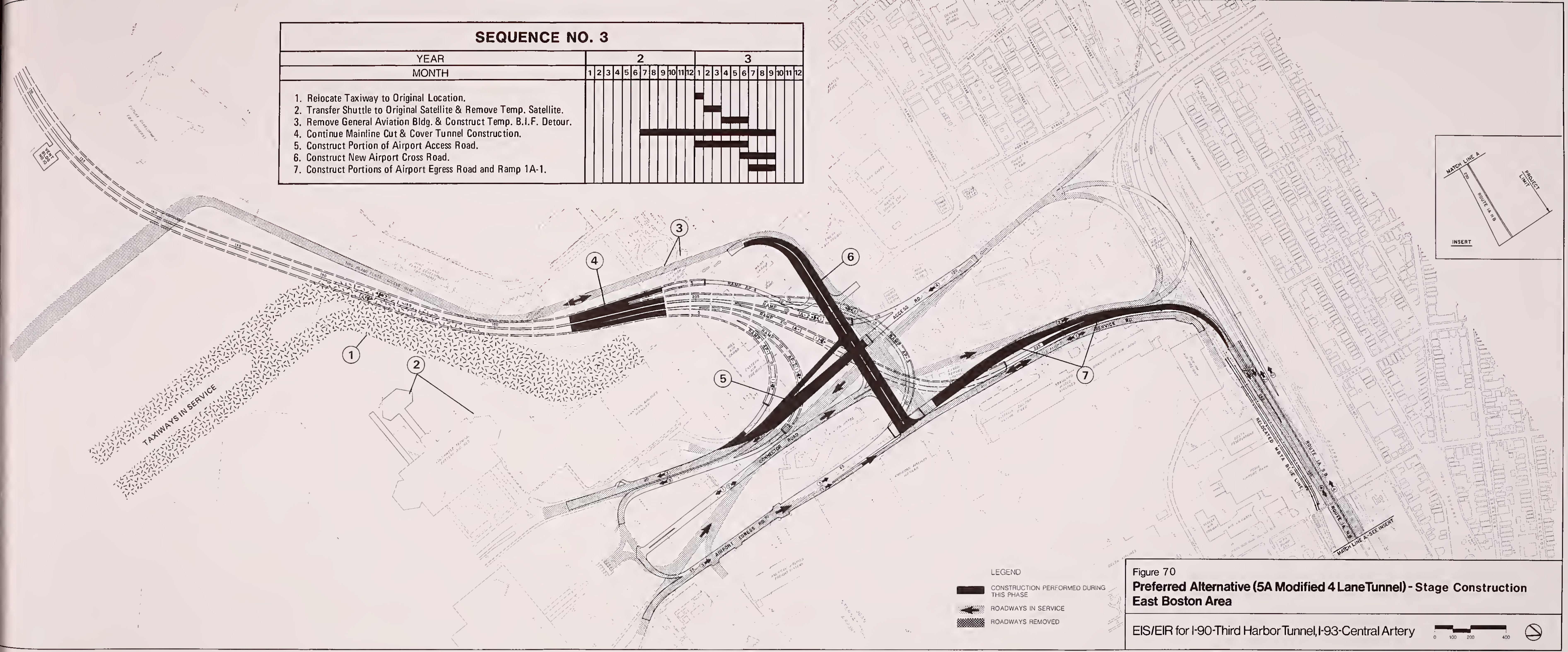


Figure 70
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

SEQUENCE NO. 4

YEAR	3												4											
MONTH	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
1. Construct Remaining Portion of Mainline Tunnel.																								
2. Place Traffic on New Cross Road.																								
3. Construct Ramp AP-3 Between Cross Road and Temp. Access Road.																								
4. Relocate Hertz Check-In.																								
5. Complete Construction of New Airport Access Road & Ramp 1A-2.																								
6. Complete Construction of New Airport Egress Road.																								
7. Complete Realignment Work of Route 1A.																								

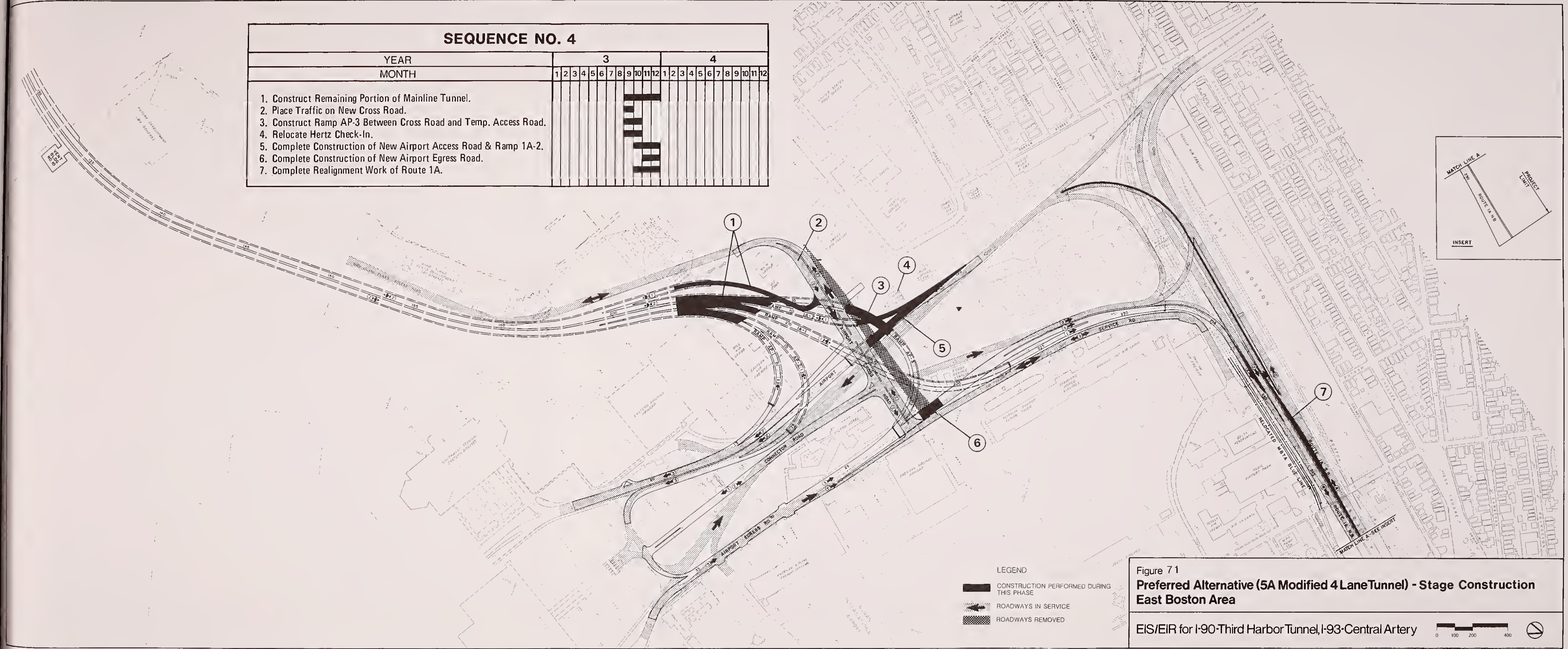


Figure 71
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

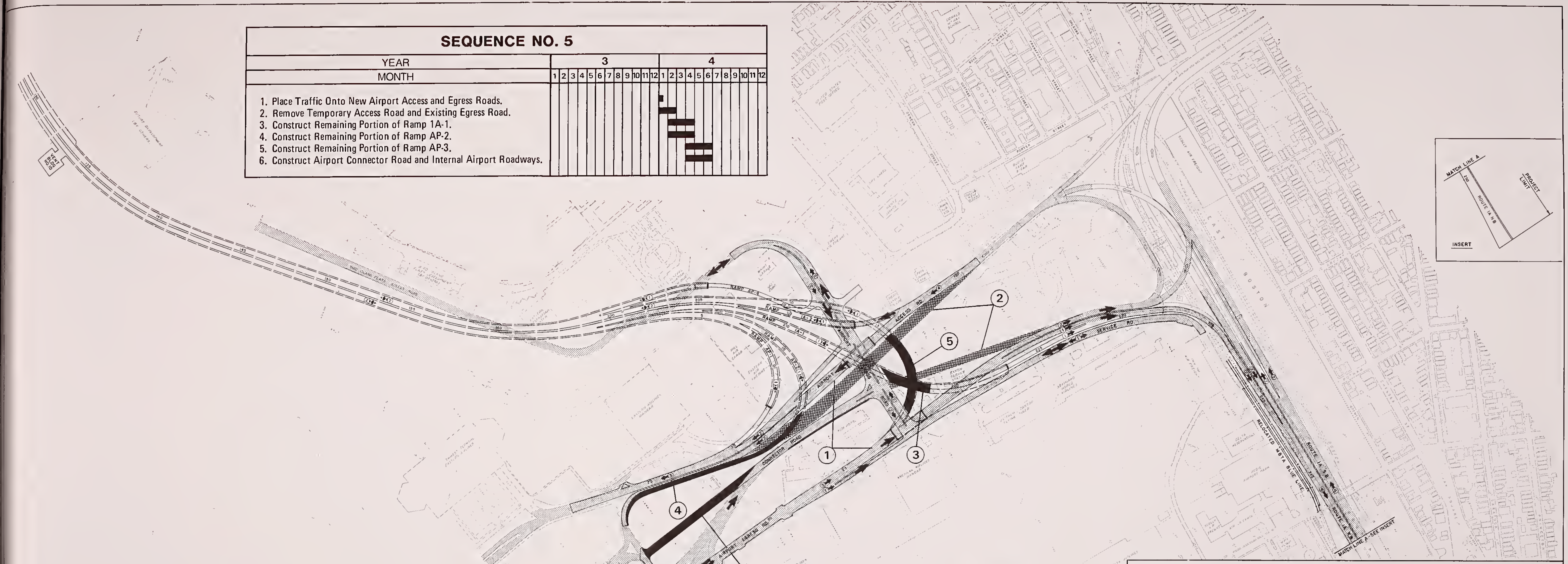


Figure 72
Preferred Alternative (5A Modified 4 Lane Tunnel) - Stage Construction East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

TIME SCHEDULE FOR CONSTRUCTION

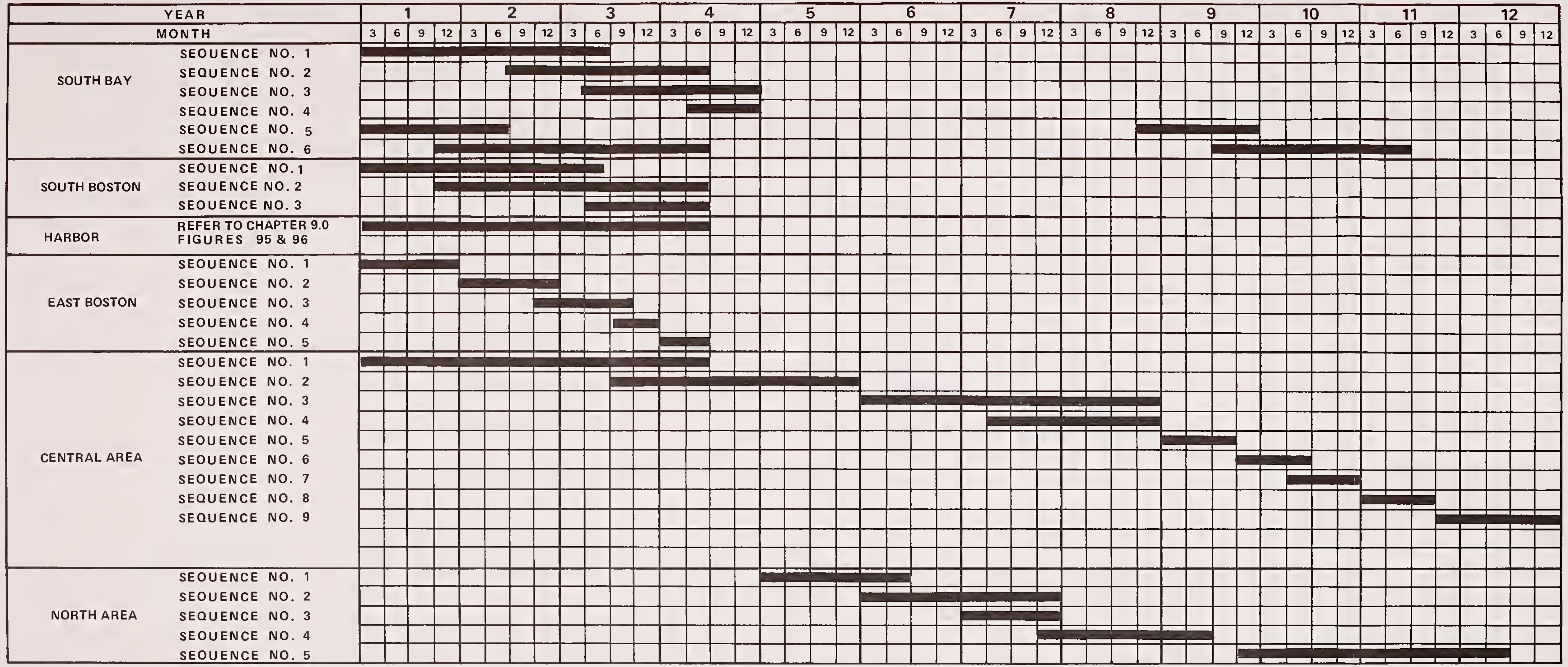
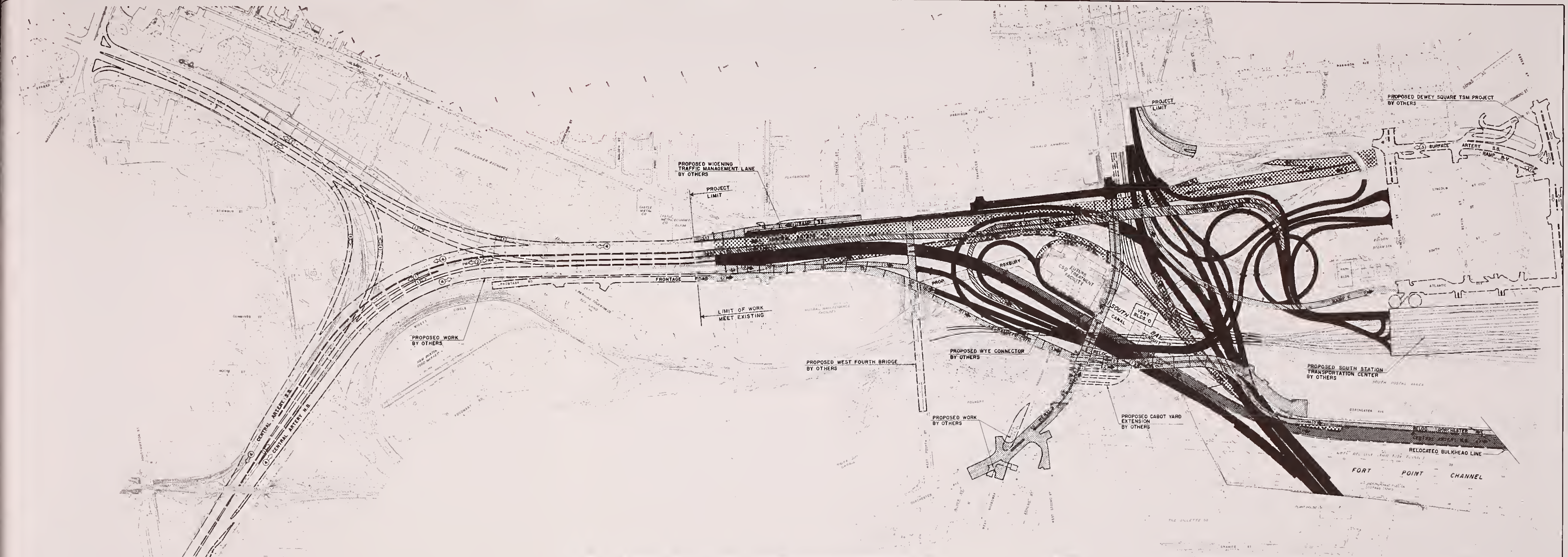


Figure 7.3
Preferred Alternative (5A Modified 4 Lane Tunnel)
Time Schedule for Construction

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



NOTES:
1. EXISTING CENTRAL ARTERY NORTHBOUND AND SOUTHBOUND WILL BE FUNCTIONAL TO YEAR 8.
2. PROPOSED NORTHBOUND ARTERY WILL BE FUNCTIONAL FROM THE END OF YEAR 8.
3. EXISTING CENTRAL ARTERY NORTHBOUND WILL CARRY CENTRAL ARTERY SOUTHBOUND TRAFFIC DURING YEAR 9.
4. AT LEAST 2 LANES OF THE SURFACE ARTERY WILL BE FUNCTIONAL AT ALL TIMES.

LEGEND
- END OF YEAR 3
- END OF YEAR 4
- END OF YEAR 8
- END OF YEAR 9
- END OF YEAR 11

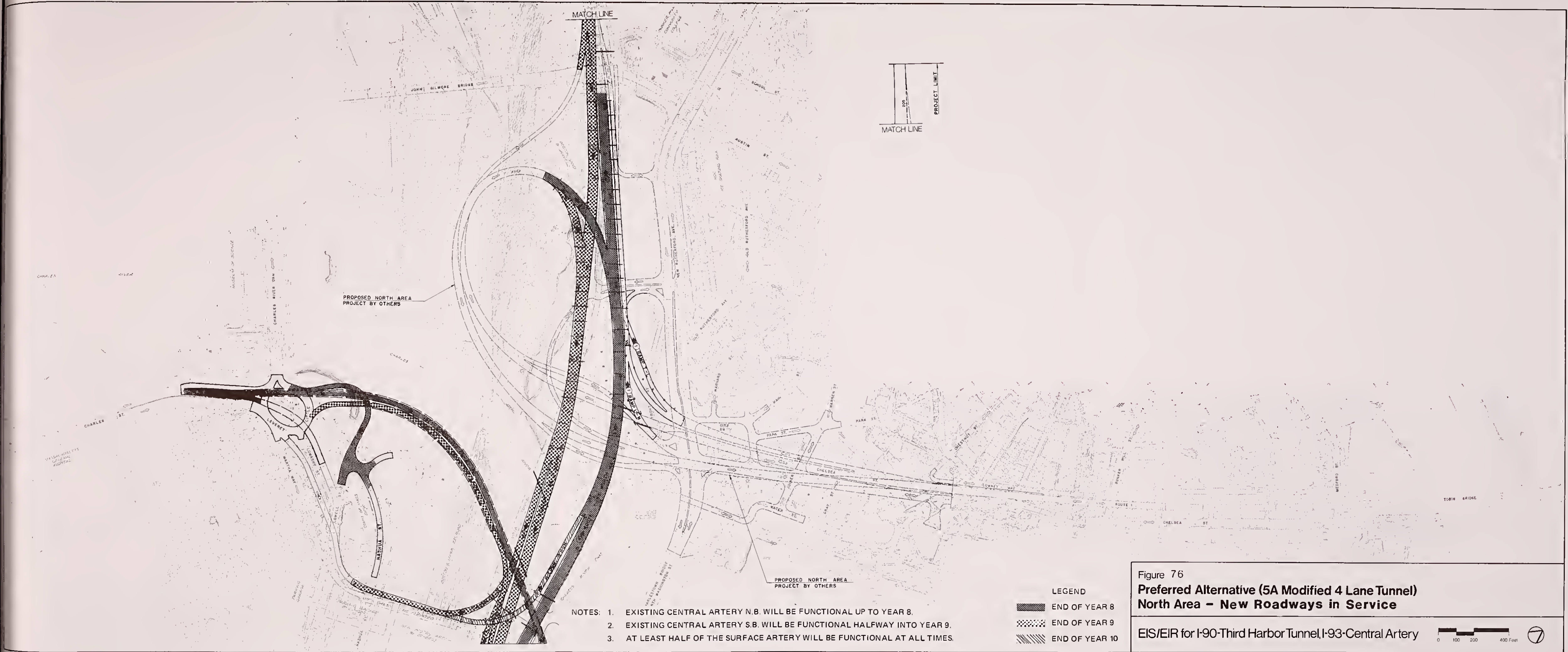
Figure 74
Preferred Alternative (5A Modified 4 Lane Tunnel)
South Bay Area - New Roadways in Service

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 100 200 400

Figure 75
Preferred Alternative (5A Modified 4 Lane Tunnel)
Central Area - New Roadways in Service

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



LEGEND
END OF YEAR 8
END OF YEAR 9
END OF YEAR 10

NOTES:
1. EXISTING CENTRAL ARTERY N.B. WILL BE FUNCTIONAL UP TO YEAR 8.
2. EXISTING CENTRAL ARTERY S.B. WILL BE FUNCTIONAL HALFWAY INTO YEAR 9.
3. AT LEAST HALF OF THE SURFACE ARTERY WILL BE FUNCTIONAL AT ALL TIMES.

Figure 76
Preferred Alternative (5A Modified 4 Lane Tunnel)
North Area – New Roadways in Service

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

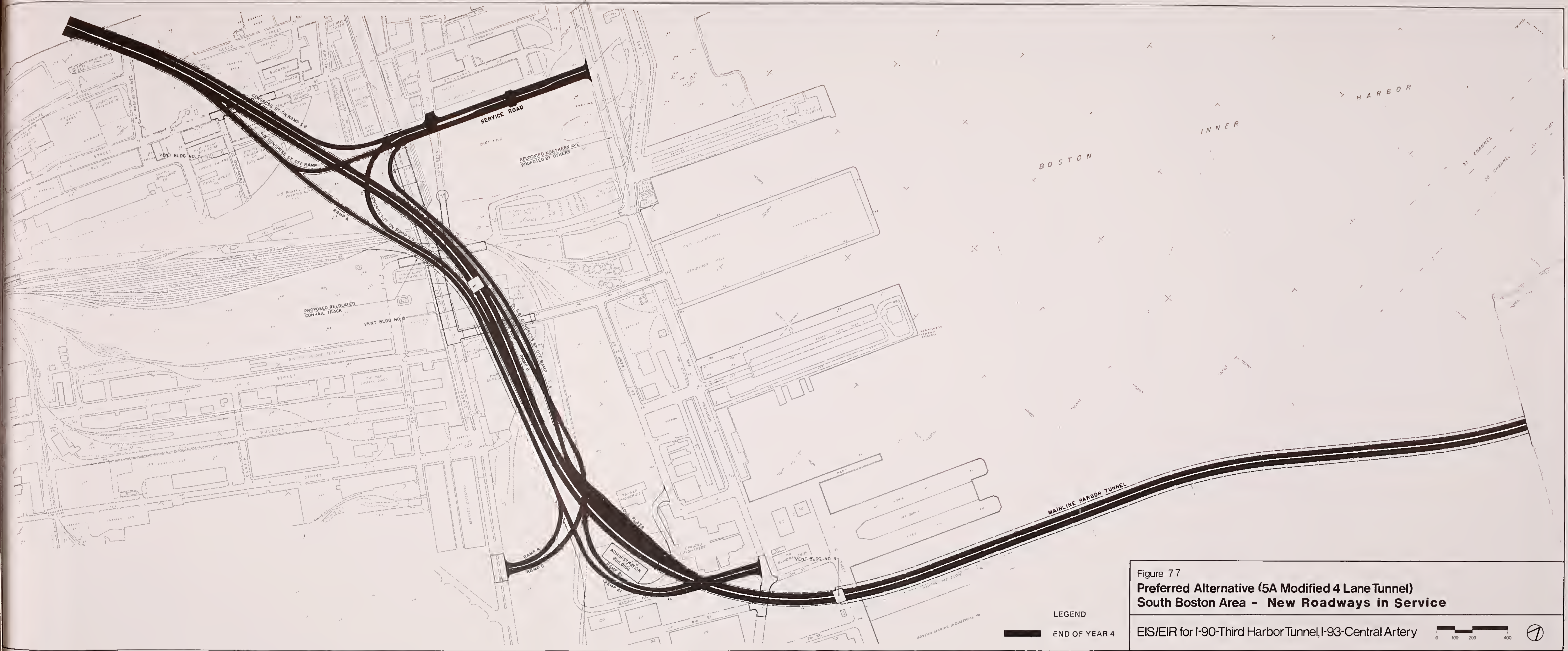


Figure 77
Preferred Alternative (5A Modified 4 Lane Tunnel)
South Boston Area - New Roadways in Service

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Figure 78
Preferred Alternative (5A Modified 4 Lane Tunnel)
East Boston Area - New Roadways in Service

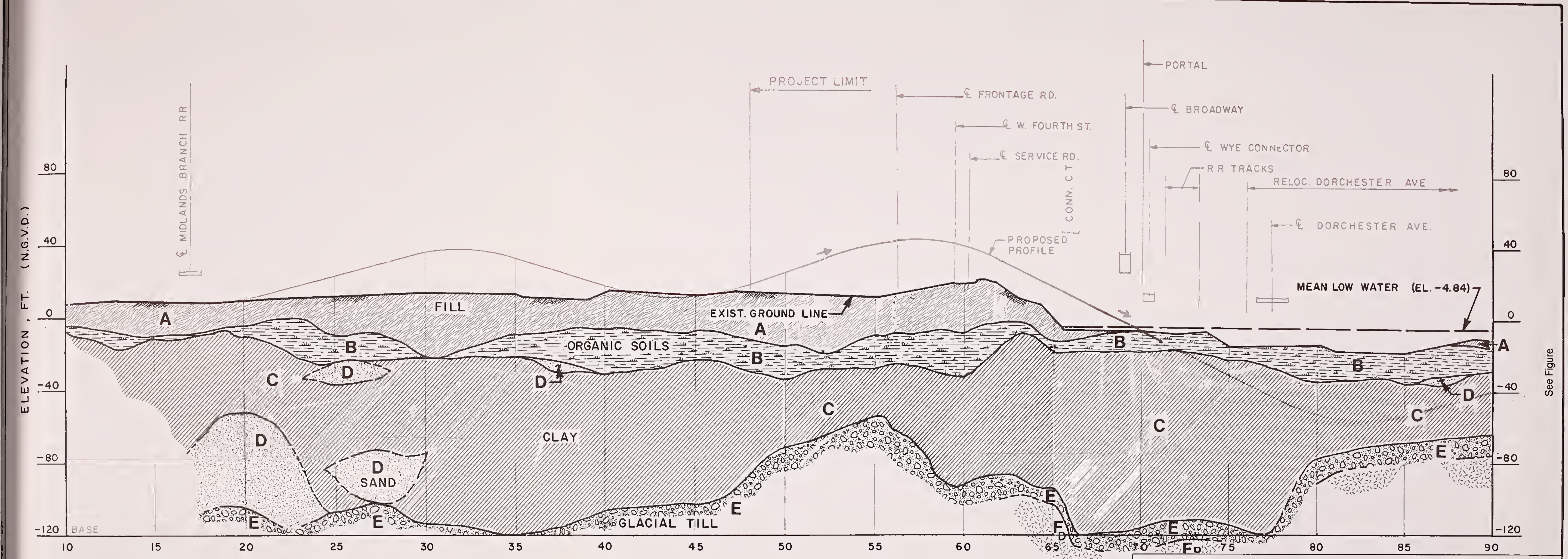
EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

LEGEND:

SB 1 DENOTES LOCATION AND NUMBER OF TEST BORING COMPLETED BY GUILD DRILLING CO., INC. SH201 AND SH202 DRILLED IN APRIL AND MAY 1982. SB1, SB2 AND SB3 DRILLED IN APRIL AND MAY 1983.

Figure 79
Preferred Alternative (5A Modified 4 Lane Tunnel)
Boring Location Plan

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



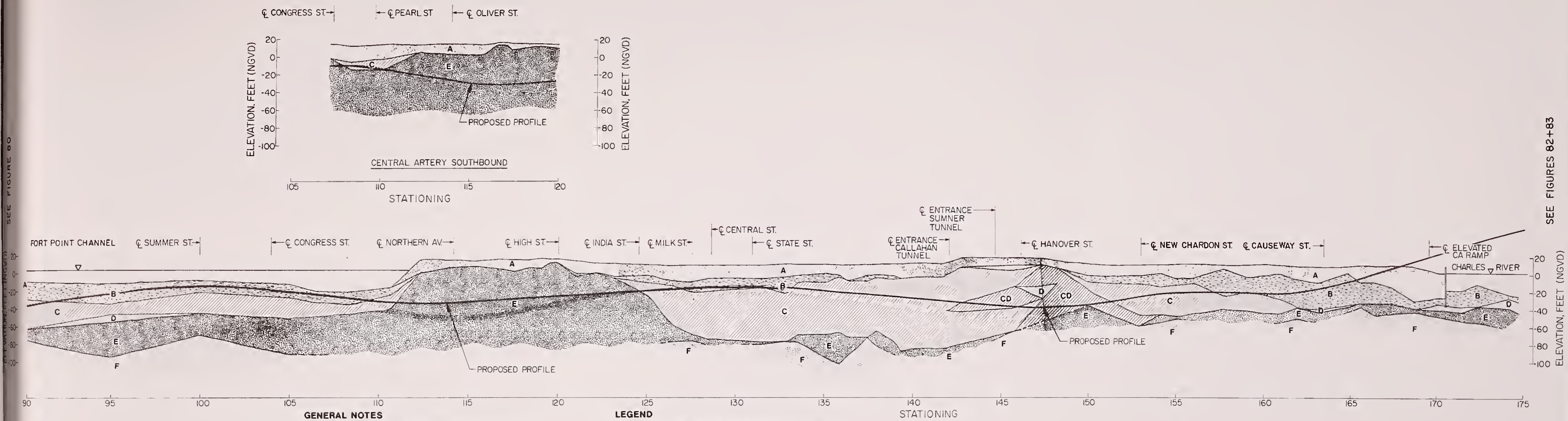
Refer to Figure 81 for General Notes and Legend.

Figure 80
Preferred Alternative (5A Modified 4 Lane Tunnel)
Subsurface Profile - South Bay Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

Horizontal Scale 0 200 400
Vertical Scale 0 20 40

HALEY & ALDRICH, INC.
CAMBRIDGE, MASSACHUSETTS



GENERAL NOTES

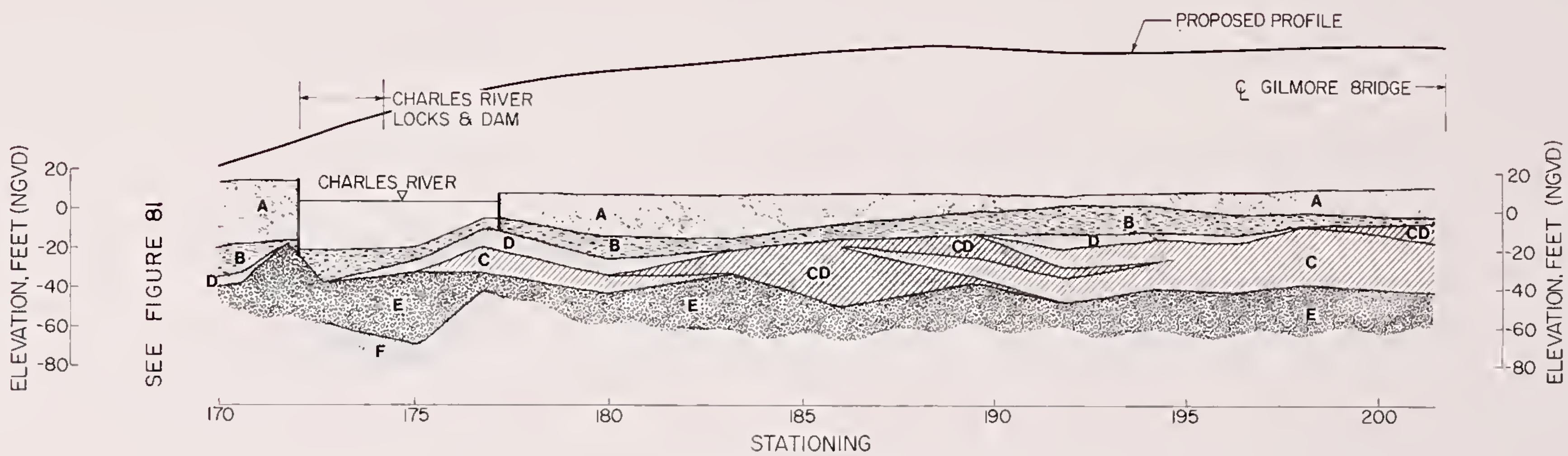
1. STRATA LINES ARE BASED UPON INTERPOLATION BETWEEN BORINGS AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIONS.
2. REFER TO THE TEXT FOR GENERALIZED DESCRIPTIONS OF SOIL AND ROCK UNITS.
3. ROAD GRADES TRACED FROM PROFILES PREPARED BY H-F-M-W.

LEGEND

SB3	DENOTES LOCATION AND NUMBER OF TEST BORING COMPLETED BY GUILD DRILLING CO., INC.
A	MISCELLANEOUS FILL
B	ORGANIC SOIL AND HARBOR SEDIMENTS
C	SILTY CLAY (MARINE DEPOSIT)
CD	CLAYEY SAND AND GRAVEL (GLACIO-MARINE DEPOSIT)
D	SAND AND GRAVEL (FLUVIAL/GLACIO-FLUVIAL DEPOSITS)
E	GLACIAL TILL
F	BEDROCK - CAMBRIDGE ARGILLITE
	BOUNDARY BETWEEN SOIL AND ROCK STRATA
	INFERRED BOUNDARY BETWEEN SOIL AND ROCK STRATA IN AREAS OF LIMITED SUBSURFACE INFORMATION.

Figure 81
Preferred Alternative (5A Modified 4 Lane Tunnel)
Subsurface Profile·Central Area

EIS/EIR for I·90·Third Harbor Tunnel, I·93·Central Artery



GENERAL NOTES

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2. REFER TO THE TEXT FOR GENERALIZED DESCRIPTIONS OF SOIL AND ROCK UNITS.
3. ROAD GRADES TRACED FROM PROFILES PREPARED BY H-F-M-W.

LEGEND

SB3 — DENOTES LOCATION AND NUMBER OF TEST BORING COMPLETED BY GUILD DRILLING CO., INC.

A — MISCELLANEOUS FILL

B — ORGANIC SOIL AND HARBOR SEDIMENTS

C — SILTY CLAY (MARINE DEPOSIT)

CD — CLAYEY SAND AND GRAVEL (GLACIO-MARINE DEPOSIT)

D — SAND AND GRAVEL (FLUVIAL/GLACIO-FLUVIAL DEPOSITS)

E — GLACIAL TILL

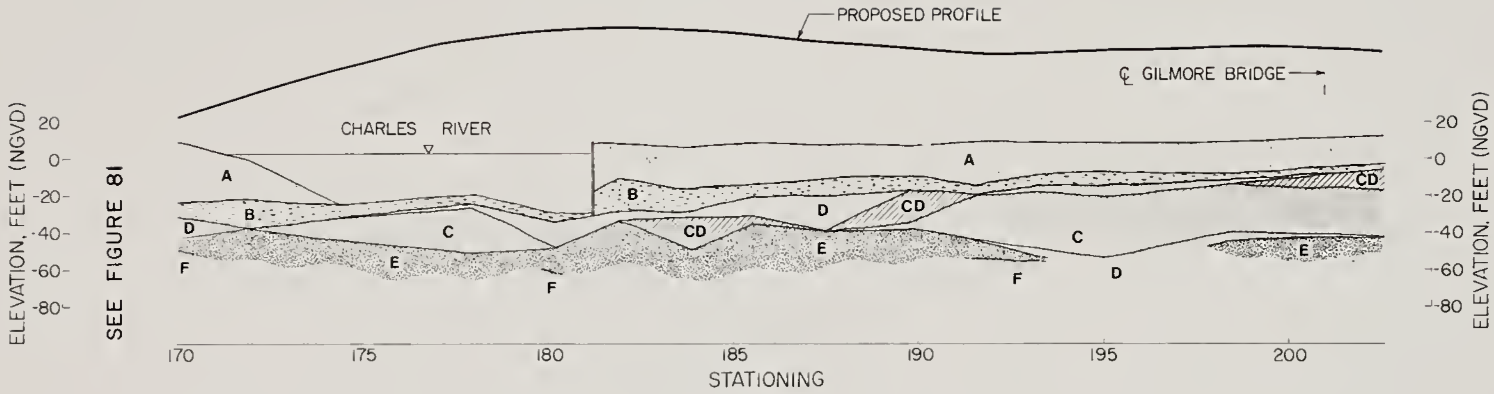
F — BEDROCK - CAMBRIDGE ARGILLITE

BOUNDARY BETWEEN SOIL AND ROCK STRATA

INFERRED BOUNDARY BETWEEN SOIL AND ROCK STRATA IN AREAS OF LIMITED SUBSURFACE INFORMATION.

Figure 82
Preferred Alternative (5A Modified 4 Lane Tunnel)
Subsurface Profile·North Area, I·93 Northbound

EIS/EIR for I·90·Third Harbor Tunnel, I·93·Central Artery



GENERAL NOTES

1. STRATA LINES ARE BASED UPON INTERPOLATION BETWEEN BORINGS AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIONS.
2. REFER TO THE TEXT FOR GENERALIZED DESCRIPTIONS OF SOIL AND ROCK UNITS.
3. ROAD GRADES TRACED FROM PROFILES PREPARED BY H-F-M-W.

LEGEND

SB3 — DENOTES LOCATION AND NUMBER OF TEST BORING COMPLETED BY GUILD DRILLING CO., INC.

A — MISCELLANEOUS FILL

B — ORGANIC SOIL AND HARBOR SEDIMENTS

C — SILTY CLAY (MARINE DEPOSIT)

CD — CLAYEY SAND AND GRAVEL (GLACIO-MARINE DEPOSIT)

D — SAND AND GRAVEL (FLUVIAL/GLACIO-FLUVIAL DEPOSITS)

E — GLACIAL TILL

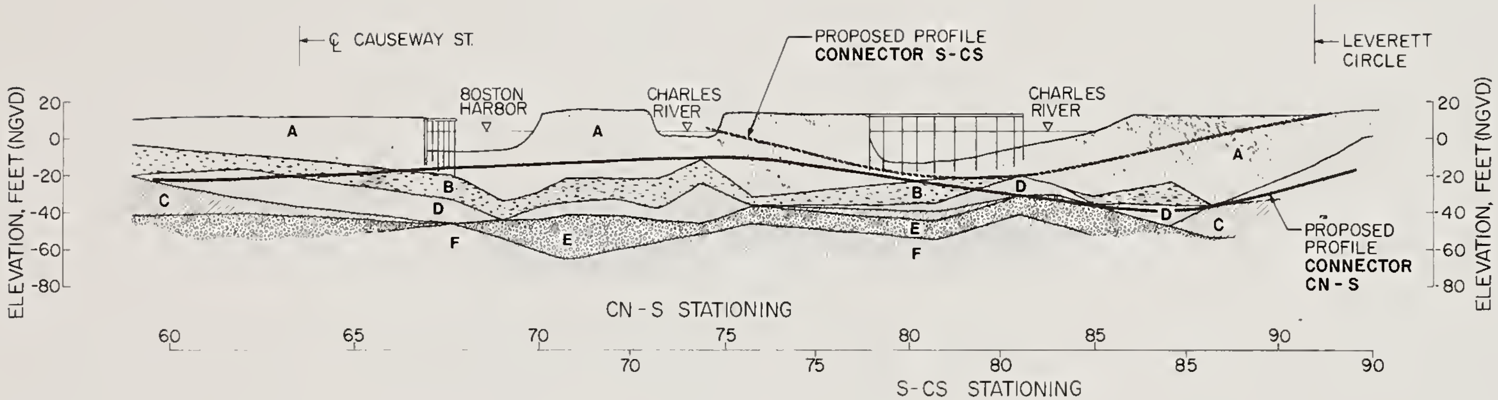
F — BEDROCK - CAMBRIDGE ARGILLITE

BOUNDARY BETWEEN SOIL AND ROCK STRATA

INFERRED BOUNDARY BETWEEN SOIL AND ROCK STRATA IN AREAS OF LIMITED SUBSURFACE INFORMATION.

Figure 83
Preferred Alternative (5A Modified 4 Lane Tunnel)
Subsurface Profile-North Area, I-93 Southbound

EIS/EIR for I-90 Third Harbor Tunnel, I-93 Central Artery



GENERAL NOTES

1. STRATA LINES ARE BASED UPON INTERPOLATION BETWEEN BORINGS AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIONS.
2. REFER TO THE TEXT FOR GENERALIZED DESCRIPTIONS OF SOIL AND ROCK UNITS.
3. ROAD GRADES TRACED FROM PROFILES PREPARED BY H-F-M-W.

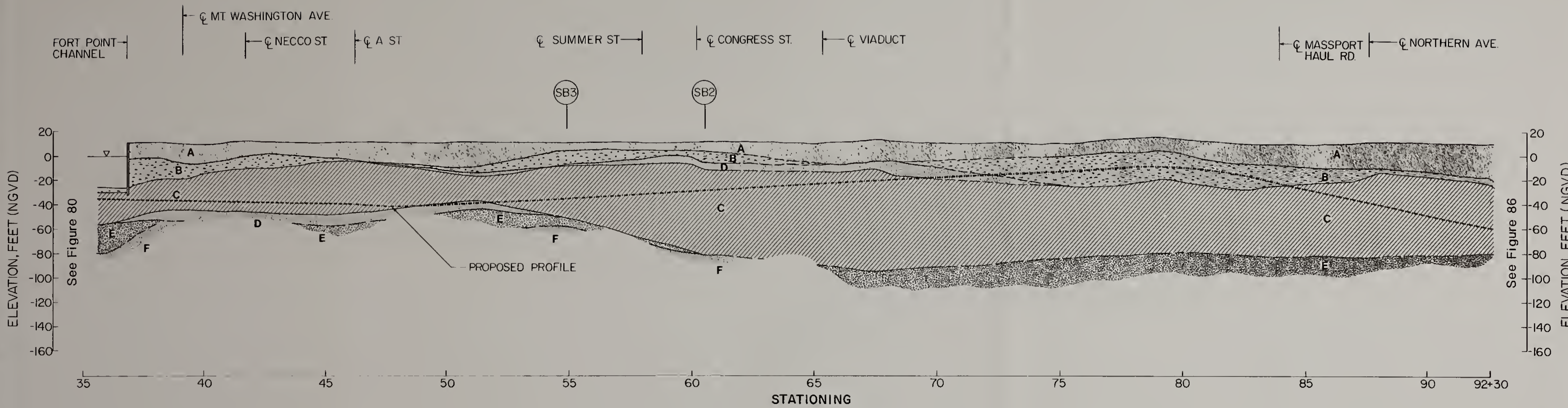
LEGEND

SB3	DENOTES LOCATION AND NUMBER OF TEST BORING COMPLETED BY GUILD DRILLING CO., INC.
A	MISCELLANEOUS FILL
B	ORGANIC SOIL AND HARBOR SEDIMENTS
C	SILTY CLAY (MARINE DEPOSIT)
CD	CLAYEY SAND AND GRAVEL (GLACIO-MARINE DEPOSIT)
D	SAND AND GRAVEL (FLUVIAL/GLACIO-FLUVIAL DEPOSITS)
E	GLACIAL TILL
F	BEDROCK - CAMBRIDGE ARGILLITE
	BOUNDARY BETWEEN SOIL AND ROCK STRATA
	INFERRED BOUNDARY BETWEEN SOIL AND ROCK STRATA IN AREAS OF LIMITED SUBSURFACE INFORMATION.

Figure 84
Preferred Alternative (5A Modified 4 Lane Tunnel)
Subsurface Profile·North Area, Leverett Circle

EIS/EIR for I·90·Third Harbor Tunnel, I·93·Central Artery

0 100 200 400 Feet



LEGEND

(SB3) DENOTES LDCATIDN AND NUMBER OF TEST BORINC CDMPLETED BY CUILD DRILLINC CO., INC.

A MISCELLANEDUS FILL

B ORCANIC SOIL AND HARBOR SEDIMENTS

C SILTY CLAY (MARINE DEPOSIT)

CD CLAYEY SAND AND CRAVEL (CLACIO-MARINE OEPOSIT)

D SAND AND CRAVEL (FLUVIAL/CLACIO-FLUVIAL DEPOSITS)

E CLACIAL TILL

F BEDROCK - CAMBRIDCE ARCILLITE

BOUNDARY BETWEEN SOIL AND ROCK STRATA

INFERRED BOUNDARY BETWEEN SOIL-AND ROCK STRATA IN AREAS OF LIMITED SUBSURFACE INFORMATION.

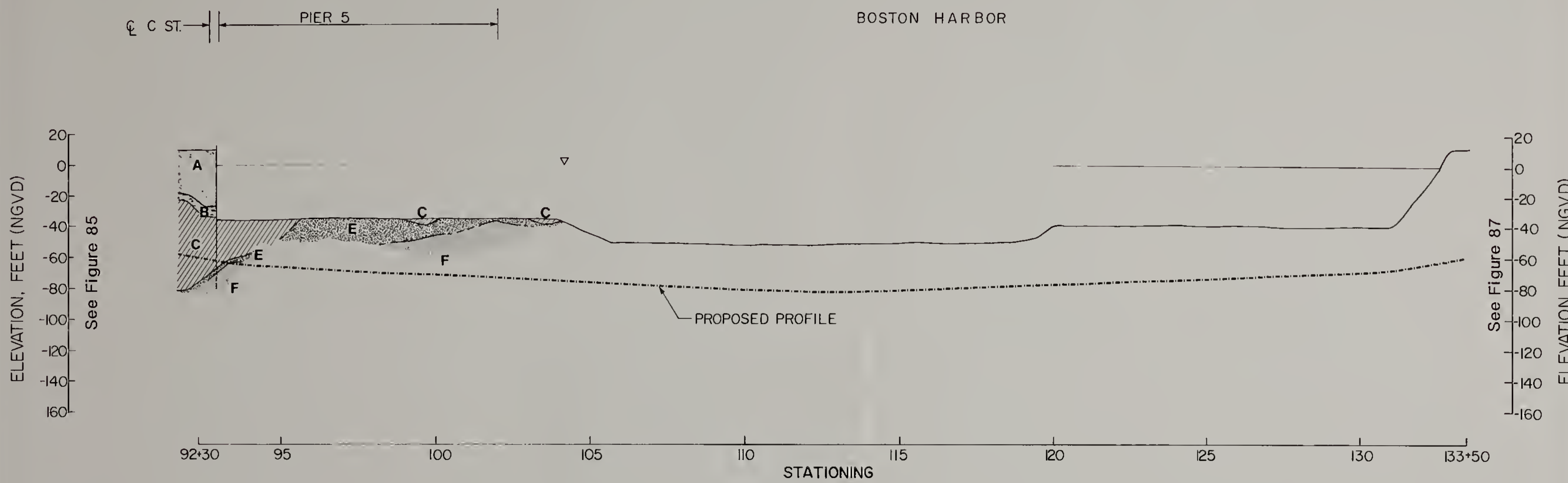
GENERAL NOTES

1. STRATA LINES ARE BASED UPON INTERPOLATION BETWEEN BORINCS AND MAY NDT REPRESENT ACTUAL SUBSURFACE CONDITIONS.
2. REFER TO THE TEXT FOR GENERALIZED DESCRIPTIONS OF SOIL AND ROCK UNITS.
3. ROAD GRADES TRACED FROM PROFILES PREPARED BY H-F-M-W.

Figure 85
Preferred Alternative (5A Modified 4 Lane Tunnel)
Subsurface Profile - South Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 100 200 400



LEGEND

- (SB3) DENOTES LOCATION AND NUMBER OF TEST BORING COMPLETED BY GUILD DRILLING CO., INC.
- A MISCELLANEOUS FILL
- B ORGANIC SOIL AND HARBOR SEDIMENTS
- C SILTY CLAY (MARINE DEPOSIT)
- CD CLAYEY SAND ANO GRAVEL (GLACIO-MARINE DEPOSIT)
- D SAND ANO GRAVEL (FLUVIAL/GLACIO-FLUVIAL OEPOSITS)
- E GLACIAL TILL
- F BEDROCK - CAMBRIDGE ARGILLITE
- BOUNDARY BETWEEN SOIL AND ROCK STRATA
- INFERRED BOUNDARY BETWEEN SOIL AND ROCK STRATA IN AREAS OF LIMITED SUBSURFACE INFORMATION.

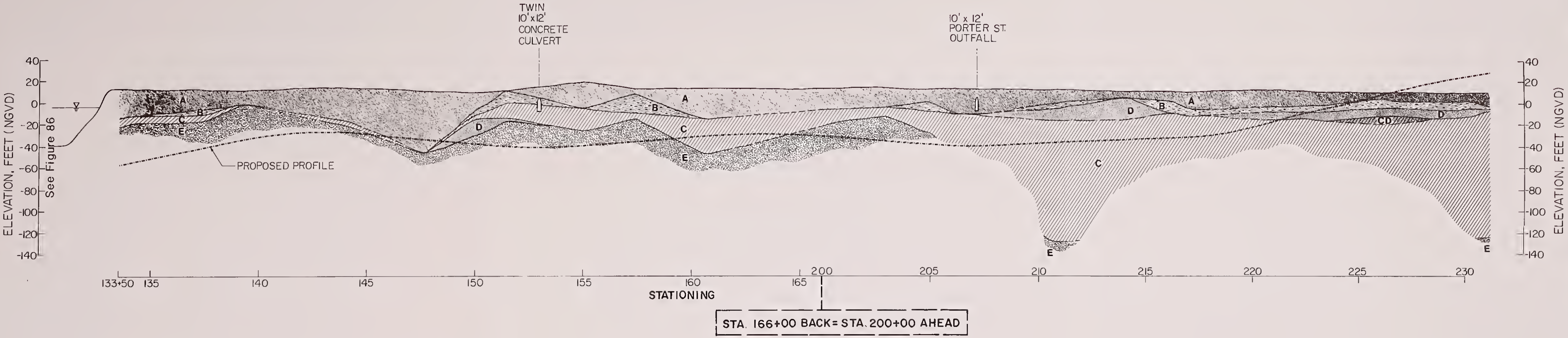
GENERAL NOTES

1. STRATA LINES ARE BASEO UPON INTERPOLATION BETWEEN BORINGS ANO MAY NOT REPRESENT ACTUAL SUBSURFACE CONOITIONS.
2. REFER TO THE TEXT FOR GENERALIZED DESCRIPTIONS OF SOIL AND ROCK UNITS.
3. ROAO GRADES TRACED FROM PROFILES PREPARED BY H-F-M-W.

Figure 86
Preferred Alternative (5A Modified 4 Lane Tunnel)
Subsurface Profile - South Boston Area (Harbor)

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 100 200 400



LEGEND

(SB3) DENDTES LDCATION AND NUMBER DF TEST BORING CDMPLETED BY GUILD DRILLING CO., INC.

A — MISCELLANEDUS FILL

B — DRGANIC SOIL AND HARBOR SEDIMENTS

C — SILTY CLAY (MARINE DEPOSIT)

CD — CLAYEY SAND AND GRAVEL (GLACIO-MARINE DEPOSIT)

D — SAND AND GRAVEL (FLUVIAL/GLACID-FLUVIAL DEPOSITS)

E — GLACIAL TILL

F — BEDROCK - CAMBRIDGE ARGILLITE

BOUNDARY BETWEEN SOIL AND ROCK STRATA

INFERRED BDUNDARY BETWEEN SOIL AND ROCK STRATA IN AREAS DF LIMITED SUBSURFACE INFORMATIDN.

GENERAL NOTES

1. STRATA LINES ARE BASED UPON INTERPOLATION BETWEEN BORINGS AND MAY NOT REPRESENT ACTUAL SUBSURFACE CONDITIDNS.
2. REFER TD THE TEXT FOR GENERALIZED DESCRIPTIDNS DF SOIL AND RDCK UNITS.
3. RDAD GRADES TRACED FRDM PROFILES PREPARED BY H-F-M-W.

Figure 87
Preferred Alternative (5A Modified 4 Lane Tunnel)
Subsurface Profile - East Boston Area

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

0 100 200 400

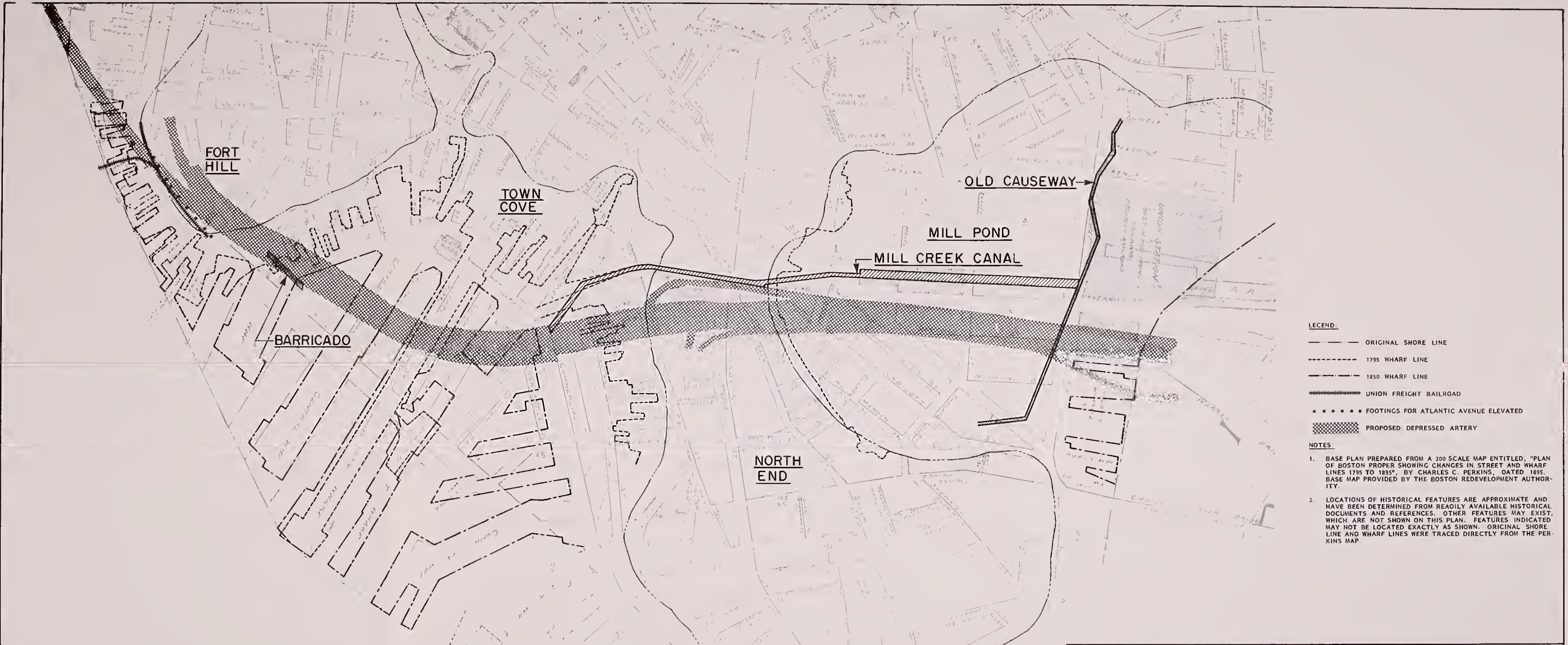
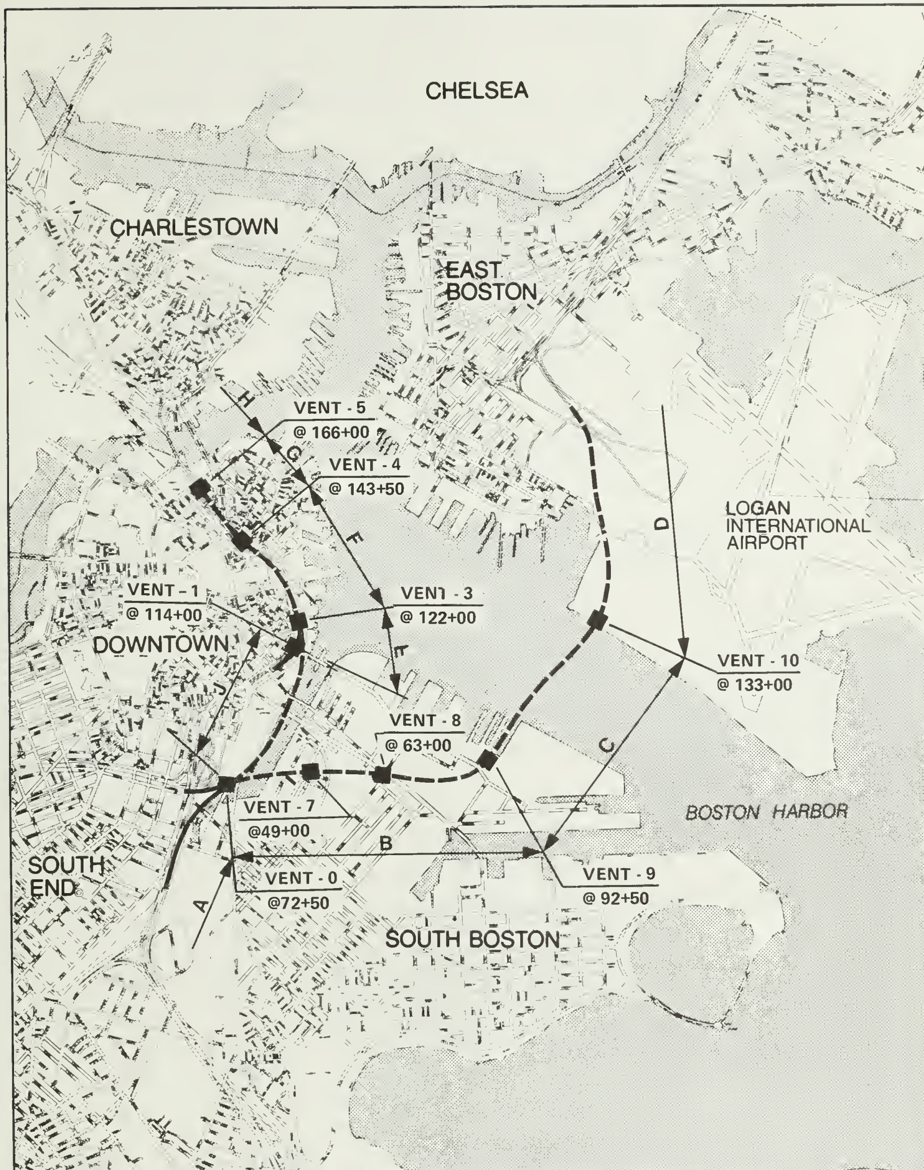


Figure 88
Original Shoreline, Wharflines and Historical Features
Central Area

EIS/EIR for I-90 Third Harbor Tunnel, I-93 Central Artery



Vent Building
A Duct Segment
Tunnel
New Roadway—at grade or elevated

Figure 89

Preferred Alternative - Ventilation

Building and Duct Segment Location

0 800 1600 3200 Feet

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

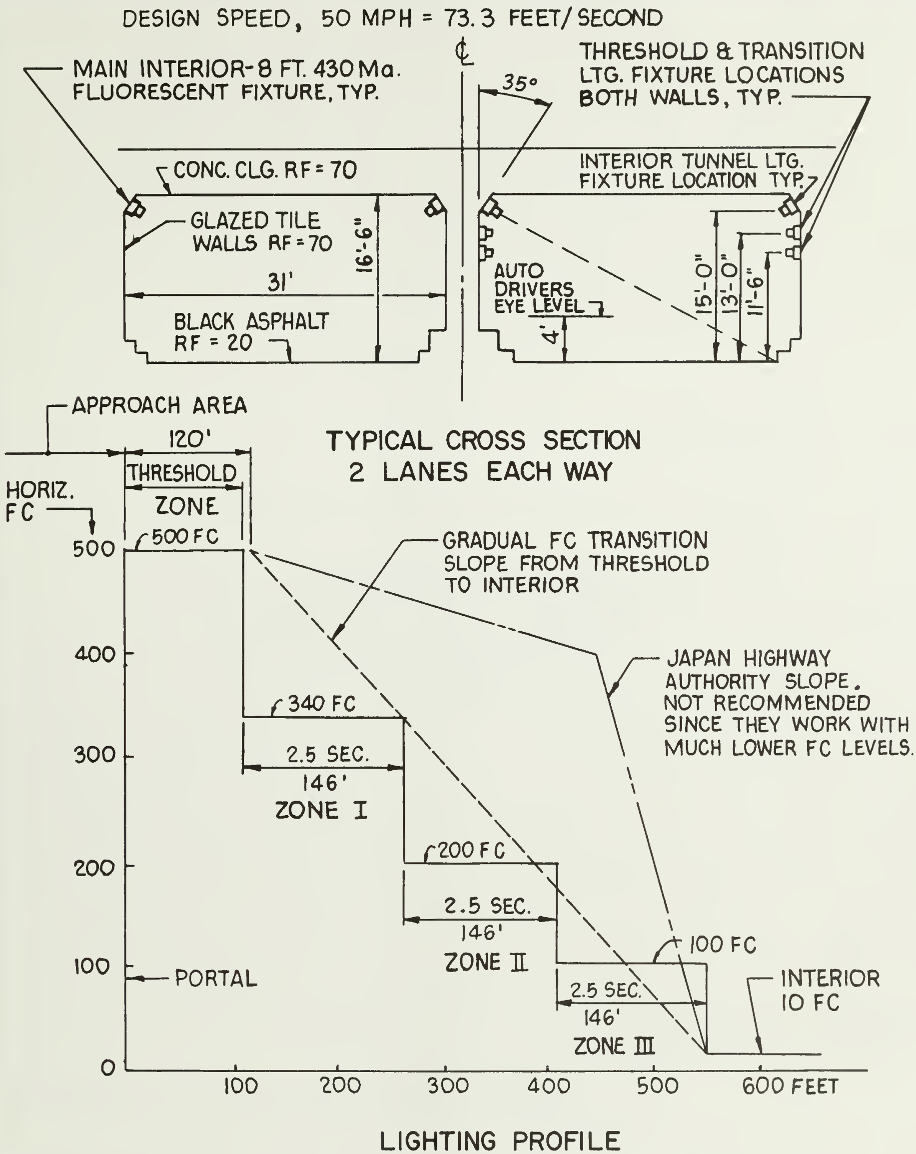
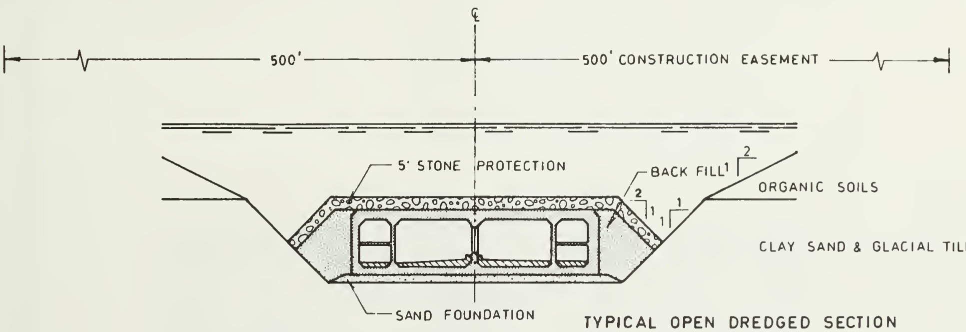


Figure 90
Tunnel Entrance Lighting Zones and Light Fixture Placement



Source: Christiani and Nielsen Corporation

Figure 91
Concrete Sunken Tube—Dredging, Typical Sections

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

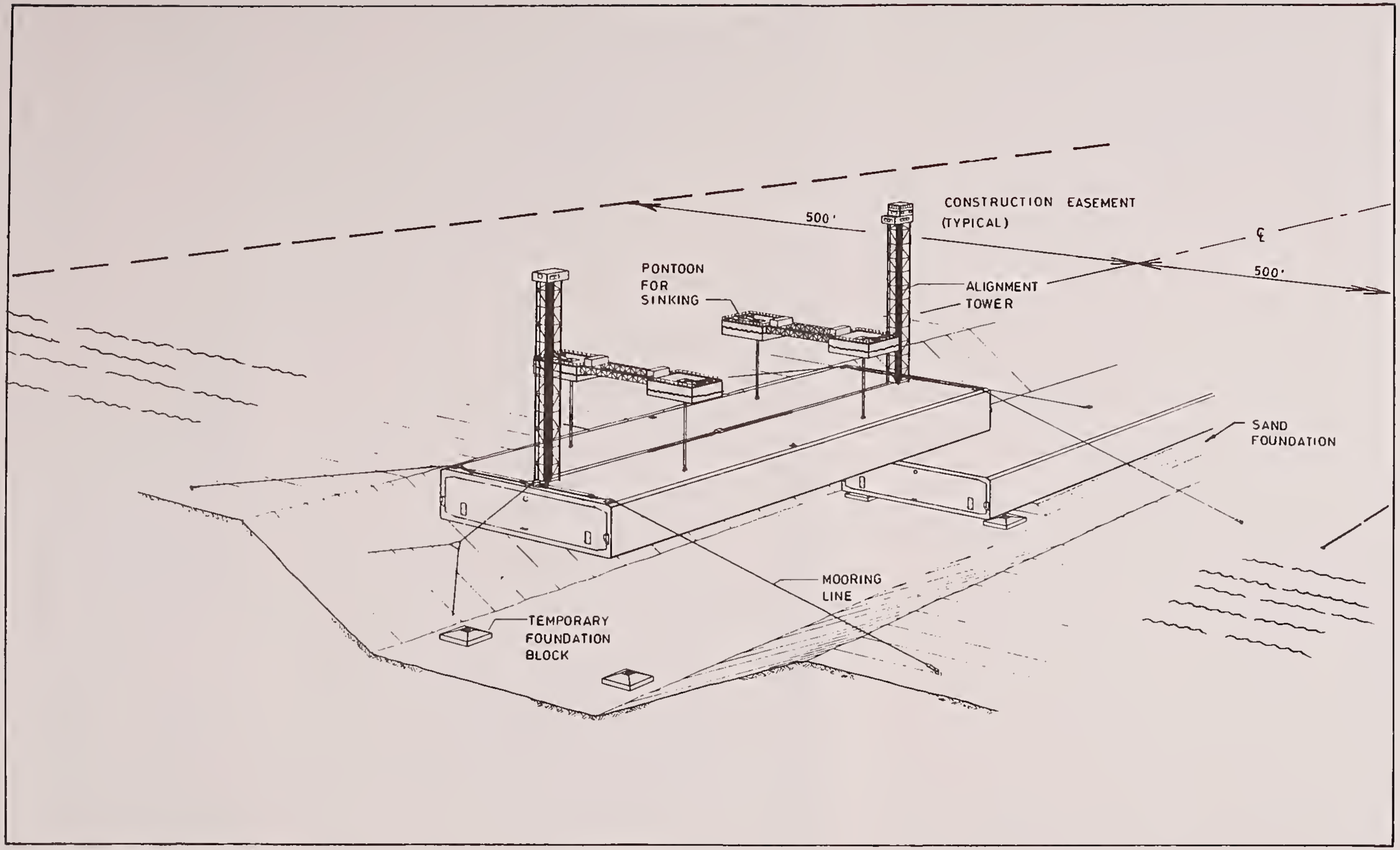
SECTION A-A (TYPICAL)
1"=80'

PLAN OF DRYDOCK
1"=800'

Source: Christiani and Nielsen Corporation

Figure 92
Concrete Sunken Tube — Drydock, Sketch Layout and Section

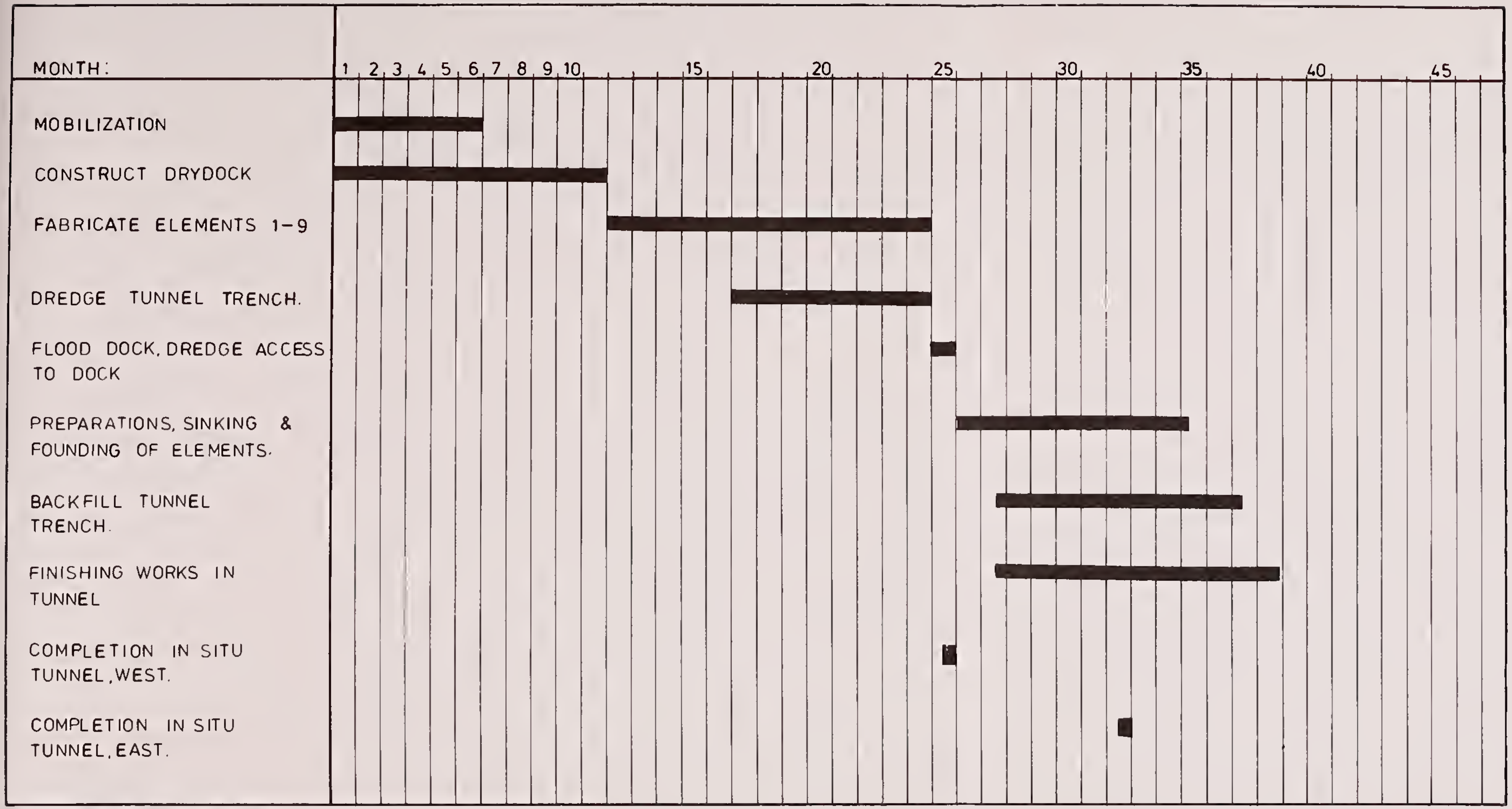
 Separate Scales for each diagram



Source: Christiani and Nielsen Corporation

Figure 93
Concrete Sunken Tube – Method of Construction

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



Source: Christiani and Nielsen Corporation

Figure 94

Concrete Sunken Tube Preliminary Construction Schedule

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery

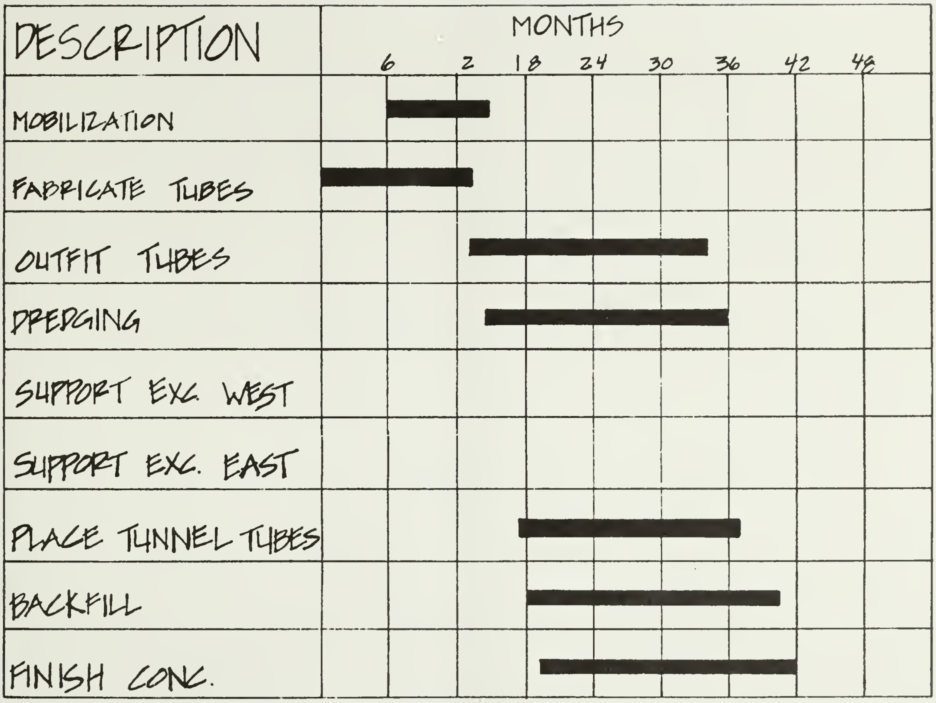
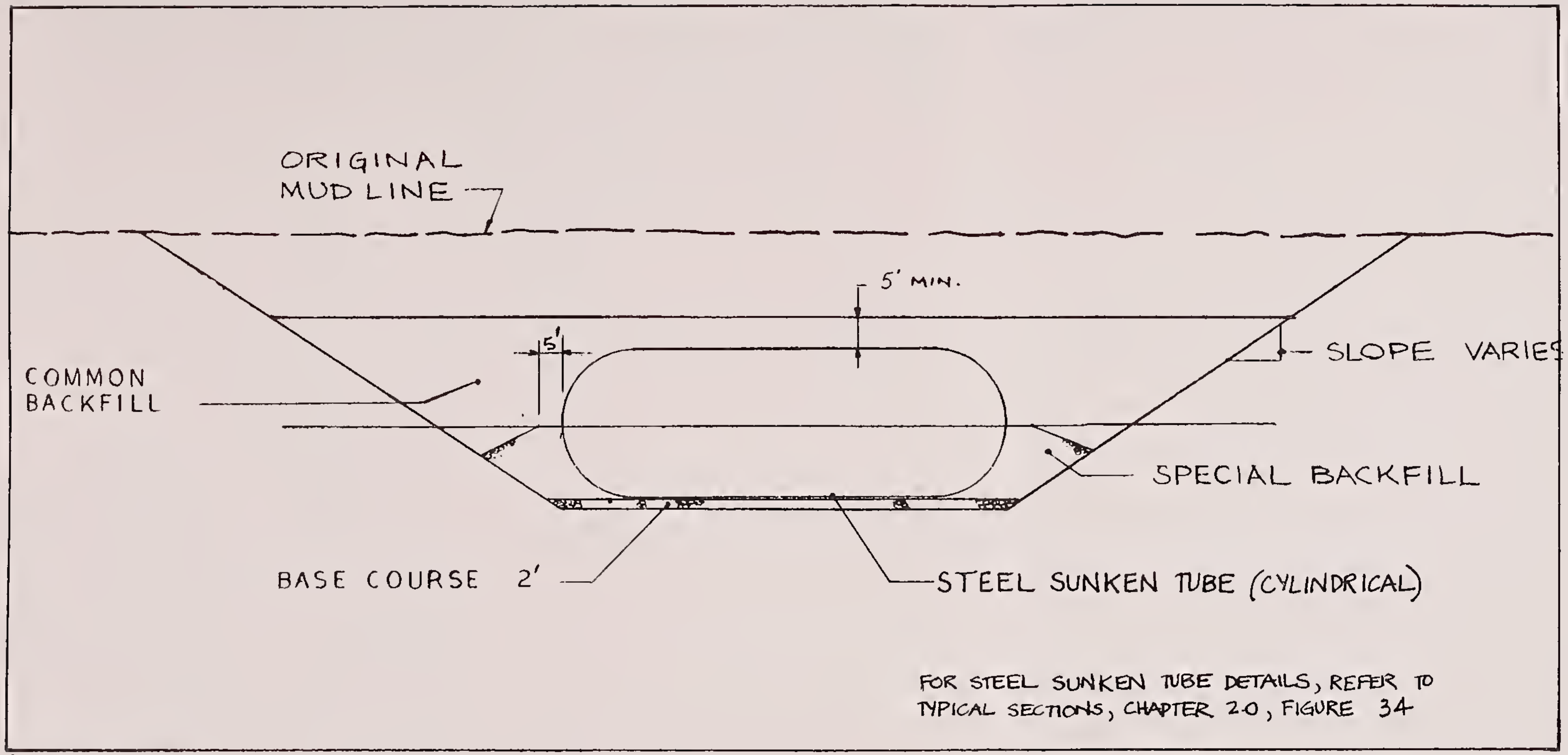


Figure 95
Steel Sunken Tube – Preliminary Construction Schedule

Source: Perini Corporation

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery



Source: Perini Corporation

Figure 96
Steel Sunken Tube
Typical Trench Section

0 30 Feet

EIS/EIR for I-90-Third Harbor Tunnel, I-93-Central Artery